Geotechnical Engineering Aspects of Trans-Tokyo Bay Highway Project

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ABSTRACT

Several significant design and construction issues related to geotechnical engineering of the Trans-Tokyo Bay Highway project are described. They are a) large scale improvement of existing soft clay deposits by in-situ cement mixing, controlling the strength of cement-mixed soft clay; b) construction of large offshore embankments by using a slurry of cement-mixed sand at the ramp sections, also controlling the strength, and dry cement-mixed sand at a flat place of the embankment; c) construction of shield tunnels with the world's largest diameter in successive stiff cement-mixed soil and very soft clay at a very shallow depth and underground connection of shield machines with the help of the ground freezing technique; and d) construction of a huge offshore diaphragm wall of 98 m in internal diameter and 119 m in height. A new method to determine the design strength of cement-mixed soil, which was adopted for the first time in this project, and a serious groundwater seepage accident in the ground inside the diaphragm wall during the construction of the Kawasaki man-made island are also described.

INTRODUCTION

The Trans-Tokyo Bay (TTB) Highway is a 15.1 km long toll highway, which links Kawasaki city with Kisarazu city across the centre of Tokyo Bay (**Fig. 1a**) (Uchida et al., 1996; JSCE et al., 1996; Tatsuoka et al., 1997). The structure comprises two 9.5 km-long shield tunnels which start from an embankment at the Ukishima Access on the Kawasaki side, pass through the Kawasaki man-made island and end at the Kisarazu man-made island, followed by a 4.4 km-long bridge extending to Kisarazu city (**Fig. 1b**). Along the route, the largest seabed depth is about 30 m and the ground condition is generally poor (**Fig. 2**). In particular, the upper part of the ground on the west side of the route is a thick very soft Holocene marine clay (Ac1, named Yurakucho Formation).

The Ministry of Construction started a feasibility survey in terms of economics and civil engineering technology in 1966. Several technical committees were organised August 1975. In August 1976, the Japan Highway Public Corporation took over the survey work. In May 1983, the Cabinet of the Japanese Government approved the construction of the highway. The Trans-Tokyo Bay Highway Corporation was established in October 1986. The actual construction started in May 1989. The ground improvement work by the sand compaction pile method and the deep mixing method (a sort of in-situ cement-mixing) started in August 1989 at several places along the route (i.e., the Ukishima Access and the Kawasaki and Kisarazu man-made islands). The construction of the foundations for the bridge started in October 1989. The construction was completed in December 1997 and the highway was opened to public on 18th December 1997.
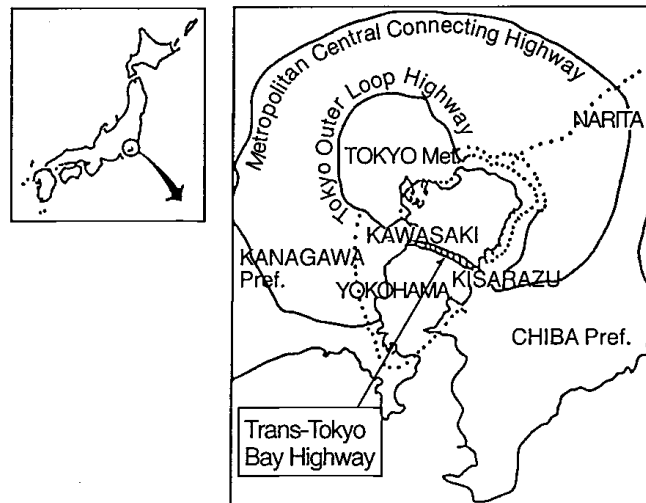
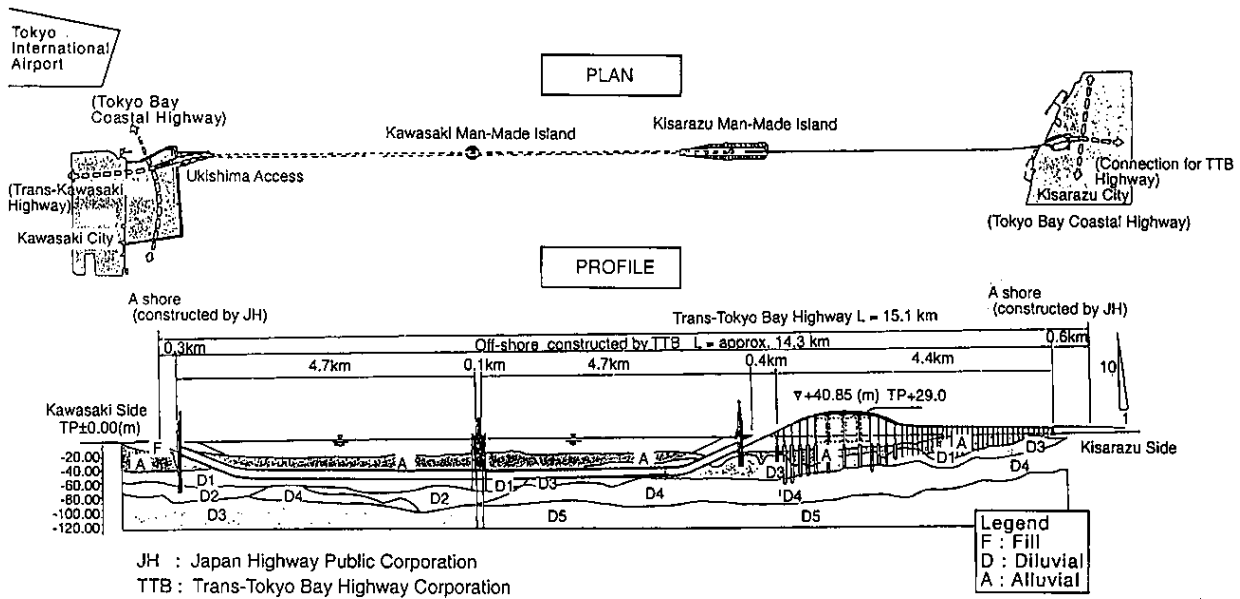Figure 1a: Location of Trans-Tokyo Bay Highway.

Figure 1b: General plan and profile of the Trans-Tokyo Bay Highway.

The construction of the Trans-Tokyo Bay Highway became technically feasible only by employing several new construction techniques related to geotechnical engineering, including:

1) large scale improvement of existing soft clay deposits by in-situ cement-mixing, controlling the strength;
2) construction of large offshore embankments using a slurry of cement-mixed sand and a dry cement-mixed sand;
3) construction of shield tunnels with the world's largest diameter in successive stiff cement-mixed soil and very soft clay at a very shallow depth and their underground connection with the help of the ground freezing technique; and
4) construction of a huge offshore diaphragm wall with the world's largest dimensions of 98 m in internal diameter and 119 m in height with the bottom end at a depth of TP-114 m (i.e., a depth below the average sea level).

This paper describes briefly the background of the employment of these techniques and outlines their procedures and results. The new concept with respect to the design strength of cement-mixed soil and a serious groundwater seepage accident that took place in the ground inside the diaphragm wall during the construction of the Kawasaki man-made island are also described.
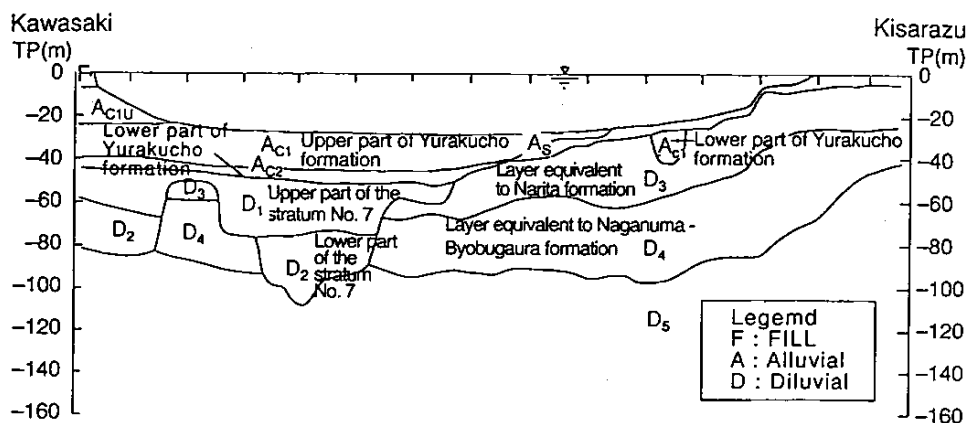

Figure 2: Soil conditions along the Trans-Tokyo Bay highway ('alluvial' and 'diluvial' mean Holocene and Pleistocene epochs, respectively).
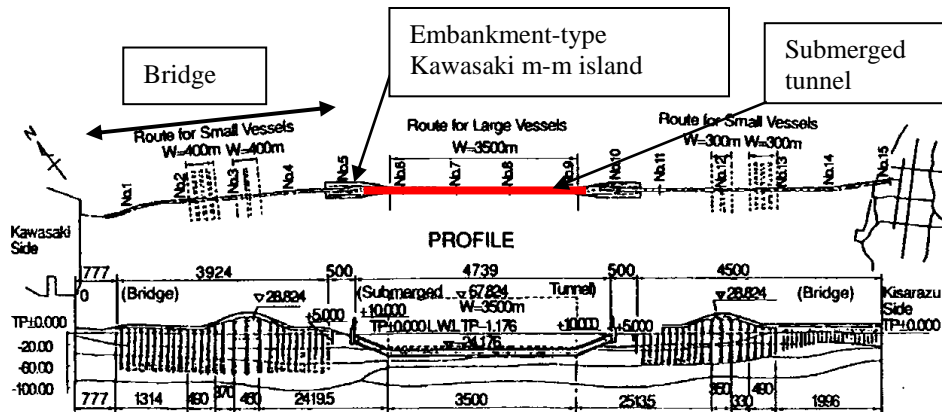
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Figure 3:   Previous proposal of the structural form of the Trans-Tokyo Bay Highway.

SELECTION OF STRUCTURAL FORM

The most difficult issue with respect to the structural form of the highway was how to safeguard extremely heavy shipping routes during construction and after completion.   The shipping flow over the planned route of the highway that was measured inOctober 1990 was 1,400 vessels per day.   Ninety percent of this traffic passes in the region of water between the Ukishima Access and the Kisarazu man-made island.   **Fig. 3** shows the proposal that was investigated most seriously among those that were proposed at early stages of investigation.   This plan was finally replaced by the final structural form.   In this previous proposal, the highway comprised submerged tunnels (i.e., immersed tube tunnels) at the central part, two bridges on both sides and two embankment-type man-made islands between them.   In 1985, this structural form was finally replaced for the following reasons (Uchida et al., 1993):

1) a bridge solution could not satisfy the required width of shipping route and cannot ensure the safety of the completed bridges;
2) a structure comprising submerged tunnels and bridges required the occupation of a large area of water for a long period during construction; and
3) it was technically difficult to keep to an acceptable level and to accurately predict the settlement of the embankment type Kawasaki man-made island, which was to be constructed on a very thick soft clay deposit, although the deposit was to be improved by a relevant method.

The present structural form, comprising the Ukishima Access, the Kawasaki island, which is a steel and RC structure, and the Kisarazu man-made island, which is basically an embankment, and shield tunnels between them, was adopted for the following reasons (Uchida et al., 1993):

1) construction of such large-diameter shield tunnels with a long tunnel drive became technically feasible (n.b., the diameter of constructed shield tunnels had reached 11.2 m);
2) construction of such a large scale diaphragm wall as the main structure of the Kawasaki man-made island became technically feasible, after similar large diaphragm walls had been constructed along the sea shore of the Tokyo Bay;
3) the present structural form has several advantages over the previous plan, in particular in that shield tunnels do not occupy the sea surface both during construction and after completion, while the excavation of seabed, which is necessary for the construction of submerged tunnels and is not environmentally favourable, becomes unnecessary; and
4) it is feasible that two tunnels (n.b., the planned final number of tunnels is three) are first constructed, while the other is constructed in the future when it becomes necessary.   This type of staged construction is not feasible with submerged tunnels.

GROUND IMPROVEMENT WORK USING NEW CEMENT-TREATMENT TECHNIQUES

Four types of ground improvement techniques by cement-treatment were used (**Table 1**).   Among those listed in Table 1, the bottom three techniques were newly developed for the project.   The total amount of cement-treated soils reached about 3.77 million m^3.   All of the ground treatment work was finished in February 1994.   The deformation and strength properties of the cement-treated soils have been reported in detail by Tatsuoka et al. (1997).   Before finally deciding to adopt the ground improvement techniques by in-situ cement-mixing and constructing fill embankments using cement-mixed soil, it was necessary to modify the conventional design method for cement-mixed soil, adapted to the large scale of the structures in this project, as described below.

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Table 1   Ground improvement techniques by cement-mixing used for the TTB Highway project

Cement-treatment method	Mixing proportion		Construction site	Volume; 1,000 m^3
Ordinary DMM	Cement W/C ratio	140 kg/m^3 100 %	Kawasaki m-m island	132
Low strength-type DMM	Cement W/C ratio	70 kg/m^3 100 %	Ukishima Access Kisarazu m-m isl. Kawasaki m-m isl.	1,248 289 168
Slurry type cement-mixed sand (*: 80 kg/m^3 in the original design)	Sand Cement Clay Sea water	1,177 kg/m^3 100 kg/m^3* 110 kg/m^3 505 kg/m^3	Ukishima Access Kisarazu m-m isl. Kawasaki m-m isl.	1,028 351 118
Dry mixture type cement-mixed sand	Sand Cement Anti-segregation adhesive	1,330 kg/m^3 100 kg/m^3 110 g/m^3	Kisarazu m-m isl.	435

Design strength of cement-mixed soil

In the conventional design method for soft clay deposits improved by in-situ mixing with cement slurry, a mass of cement-mixed soil is treated like a mass of unreinforced concrete (such as a gravity-type concrete dam).   The stability is examined by comparing the working stress, which is usually evaluated based on the elastic theory, with the allowable stress (i.e., the allowable stress method).   The allowable stress is usually specified to be equal to 1/6 - 1/5 of the unconfined compressive strength, q_u.   This small fraction of the peak strength has been used based on the observation that cement-mixed soil fails in a very brittle manner in the unconfined compression test, exhibiting essentially zero residual strength, and the consideration that the use of peak strength is very unconservative for such a brittle material.

At the initial design stage of this project, the allowable stress method was adopted to examine the seismic stability of the ramp sections, comprising existing soft clay deposits improved by the DMM and fill embankments constructed by using a slurry of cement-mixed sand, at the Ukishima and Kisarazu man-made islands.   The results from a series of
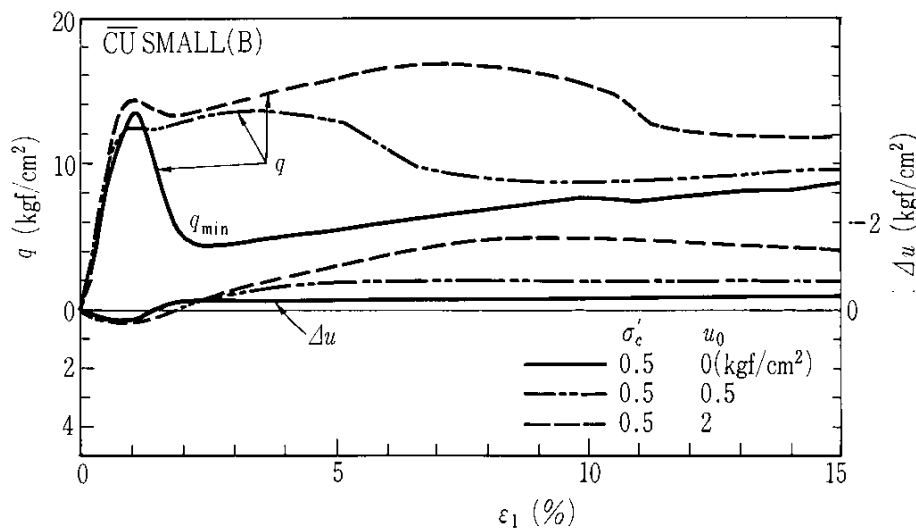
Figure 4:   Effects of back pressure on the stress-strain properties of cement-mixed sand (slurry type) from CU TC tests on specimens produced in the laboratory (Hashimoto et al., 1994).

seismic stability analyses showed that the expected working stress would exceed the allowable stress, indicating that the structures would lose the stability during anticipated earthquakes.   However, this prediction was considered not realistic and too conservative.

After a series of investigations on this issue was performed (Tatsuoka and Kobayashi 1983; Tatsuoka et al., 1997), the following conclusions were obtained:
1) The conventional allowable stress design method would be relevant to the following cases:
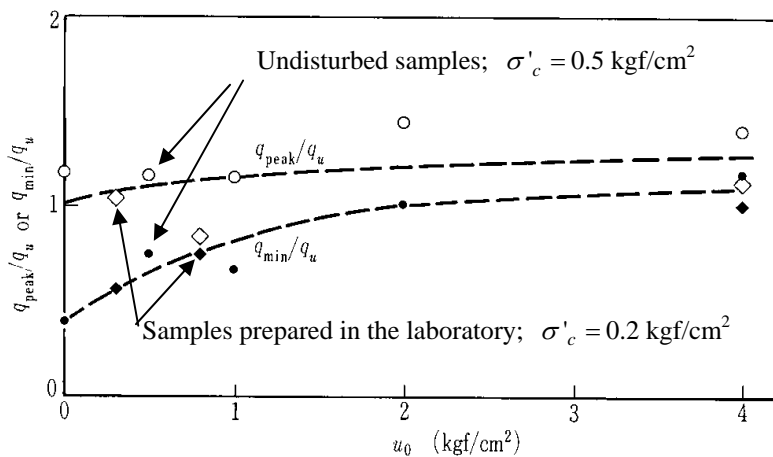
Figure 5: Effects of back pressure on the peak and residual strengths of cement-mixed sand from CU TC tests on specimens produced in the laboratory and those retrieved from a field full-scale test (Hashimoto et al., 1994).

a) failure occurs essentially under drained conditions with zero or very low confining pressure; and/or

b) a mass of cement-mixed soil is relatively small while the cement-mixed soil is rather rigid and strong in comparison to the adjacent soil, such as an untreated soft clay deposit.

2) On the other hand, in the present project:

a) majority of the cement-mixed soil is confined by the adjacent existing ground under water; and

b) the dimensions of cement-mixed soil masses are huge, much larger than those ever constructed, and the cement-mixed soil is made intentionally relatively soft and weak (as explained later).

3) For geotechnical engineering stability analysis of a soil mass, the limit equilibrium-based stability approach is usually employed on the premise that the failure of soil mass is not brittle, not as the failure behaviour of cement-mixed sand in the unconfined compression tests. It was decided to adopt this conventional geotechnical engineering design approach for the stability analysis of the cement-mixed soil masses in this project, while taking into account the following specific features:
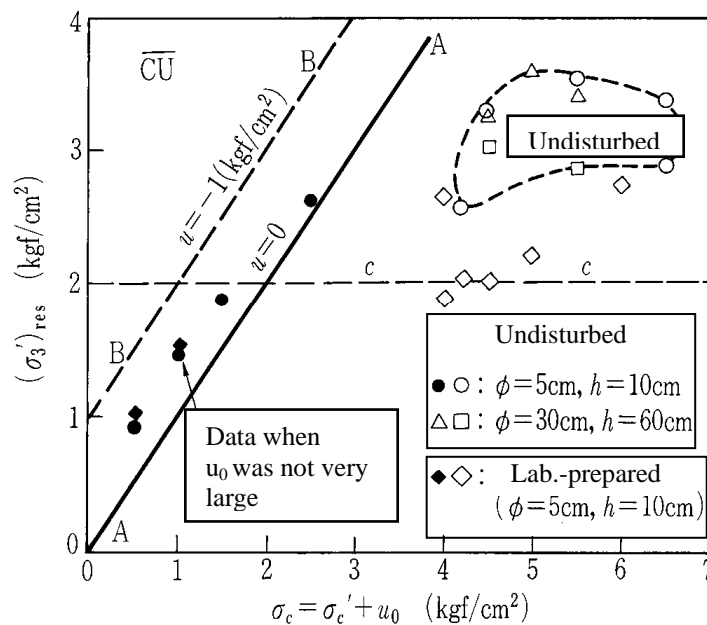
Figure 6: Relationships between the effective confining pressure at the residual state and the initial total confining pressure for cement-mixed sand slurry from CU TC tests on specimens produced in the laboratory and those retrieved from a field full-scale test (Hashimoto et al., 1994).

a) failure of a cement-mixed soil mass would be progressive in the sense that the respective peak strength at each point along the failure plane is not mobilised simultaneously, but the operating average strength should be considerably smaller than the average peak strength; and

b) the drainage conditions for a submerged mass of cement-mixed soil should be either 'drained' in long-term stability analysis or 'undrained' in seismic stability analysis.

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It was considered that the effects of progressive failure could be taken into account duly and in a conservative way by using the residual strength, which is mobilised at large strains in the post-peak regime, along the whole failure plane in the limit equilibrium-based stability analysis. The deformation of the ground and the structure during the design earthquake was evaluated separately by the equivalent linear FEM analysis taking into account the dependency of the stiffness on strain level (Uchida et al., 1993; Tatsuoka et al., 1997).

Effects of drainage conditions (or effects of possible build-up of excess pore pressure) and initial confining pressure (or initial hydrostatic water pressure) on the peak strength of cement-mixed soil are not significant in this case. On the other hand, these effects on the residual strength are significant, which were therefore taken into account in the design, as shown below. **Fig. 4** shows the effects of the initial pore water pressure u_0 (or the back pressure), which is equivalent to the hydrostatic pressure in the field, on the stress-strain behaviour of the cement-mixed sand, obtained from a series of consolidated undrained (CU) triaxial compression (TC) tests. The specimens (5cm in diameter and 10 cm in height) were prepared in the laboratory using the mixing proportion for a slurry type cement-mixed sand employed in the design (see Table 1). The specimens were consolidated to the same isotropic confining pressure $\sigma'_c = 0.5$ kgf/cm^2 (49 kPa). In the test with zero u_0, in the post-peak regime, the q value started increasing again after having exhibited a temporary minimum value q_{min}. As this behaviour is due to a kinematic restraint due to the friction at the specimen ends, such an increase in q cannot be expected in the field. The values of q_{peak} (peak strength) and q_{min} (as the residual strength) were normalised by the corresponding unconfined compressive strength q_u and have been plotted against u_0 in **Fig. 5**. The following trends of behaviour may be seen from Figs. 4 and 5:

1) the peak strength q_{peak} is rather independent of back pressure u_0; and

2) the residual strength q_{min} becomes larger with the increase in the back pressure u_0.

The second observation can be explained by the effects of u_0 on the effective confining pressure at the residual stress $(\sigma'_3)_{res}$, which is obtained as:

$$(\sigma'_3)_{res} = \sigma'_c - (\Delta u)_{res} = \sigma'_c - \{(u)_{res} - u_0\} = \sigma_c - (u)_{res} \tag{1}$$

The pore water pressure at the residual state $(u)_{res}$ cannot become lower than –1.0 kgf/cm^2 (98 kPa), so $(\sigma'_3)_{res}$ cannot become larger than σ_c $(= \sigma'_c + u_0)$ + 1.0 kgf/cm^2 (98 kPa). This relationship is represented by line B-B in **Fig. 6**. The data from a series of CU TC tests performed on specimens prepared in the laboratory and undisturbed samples retrieved from a large scale field test, both having the same mixing proportion, are plotted in this figure. The observed lowest values of $(u)_{res}$ was about – 0.5 kgf/cm^2 (- 50 kPa) due to early cavitation of gas that had been dissolved in the samples. Eq. 1 also indicates that for the same initial effective confining pressure σ'_c, the value of $(\sigma'_3)_{res}$ may increase with the increase in σ_c $(= \sigma'_c + u_0)$; i.e., with the increase in u_0. This trend can be seen when σ_c is relatively low in Fig. 6. This means that for the same initial effective stress σ'_c, the undrained shear strength of cement-mixed soil, which is essentially proportional to $(\sigma'_3)_{res}$, may increase with the increase in the depth of water. It may be seen from Fig. 6, however, that the observed values of $(\sigma'_3)_{res}$ have an upper-bound value. The upper bound is controlled by the intrinsic dilatancy properties, among others, of the material. In the design, an upper bound equal to 2.0 kgf/cm^2 (196 kPa) (line C-C) was employed based on the data presented in Fig. 6.

Based on the laboratory test results described above, the design shear strength of the cement-mixed soil was determined as follows:

a) <u>Drained shear strength $(\tau_f)_d$</u> to be used for long-term stability analyses:

$$(\tau_f)_d = (\sigma'_0 + \Delta\sigma') \cdot \tan\phi'_{res} \tag{2}$$

where $(\tau_f)_d$ is the design drained residual shear strength; σ'_0 is the initial effective normal stress acting on the failure plane; $\Delta\sigma'$ is the increase in the effective normal pressure σ' by loading under drained conditions; and ϕ'_{res} is the angle of internal friction at the residual condition, which was equal to 40° (with zero cohesion intercept) in this project.

b) <u>Undrained shear strength $(\tau_f)_u$</u> to be used for seismic stability analyses:

$$(\tau_f)_u = Min.(\tau_1, \ \tau_2) \tag{3a}$$

$$\tau_1 = (\sigma)_{res} \cdot \tan\phi'_{res}; \ \ \tau_2 = (\sigma')_{res} \cdot \tan\phi'_{res} \tag{3b}$$

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where $(\tau_f)_u$ is the design undrained residual shear strength. τ_1 is the undrained residual shear strength that is controlled by the hydrostatic pressure u_0. The use of τ_1 is relevant for cement-mixed soil at relatively shallow depths of water. $(\sigma)_{res}$ is the total normal pressure at the residual condition acting on the failure plane, which is specified to be equal to the initial total normal pressure σ, assuming that $(u)_{res} = 0$, as a conservative decision. τ_2 is the undrained shear strength that is controlled by the intrinsic value $(\sigma')_{res}$ of the material, independent of $(\sigma)_{res}$. The use of τ_2 is relevant for cement-mixed soil at relatively large depths of water. For the cement-mixed soil located at depths larger than a certain value, the undrained shear strength becomes larger than the corresponding drained shear strength. Results from seismic stability analysis showed that even by using the drained shear strengths, which were basically smaller than the corresponding undrained shear strength in this case, an ample factor of safety could be ensured against the design seismic load. This means that the mass of cement-mixed soil that had been judged to be unstable during the design earthquakes based on the conventional allowable stress method was judged to be safe based on the limit equilibrium stability method using the proposed design shear strength (i.e., Eqs. 2 and 3).
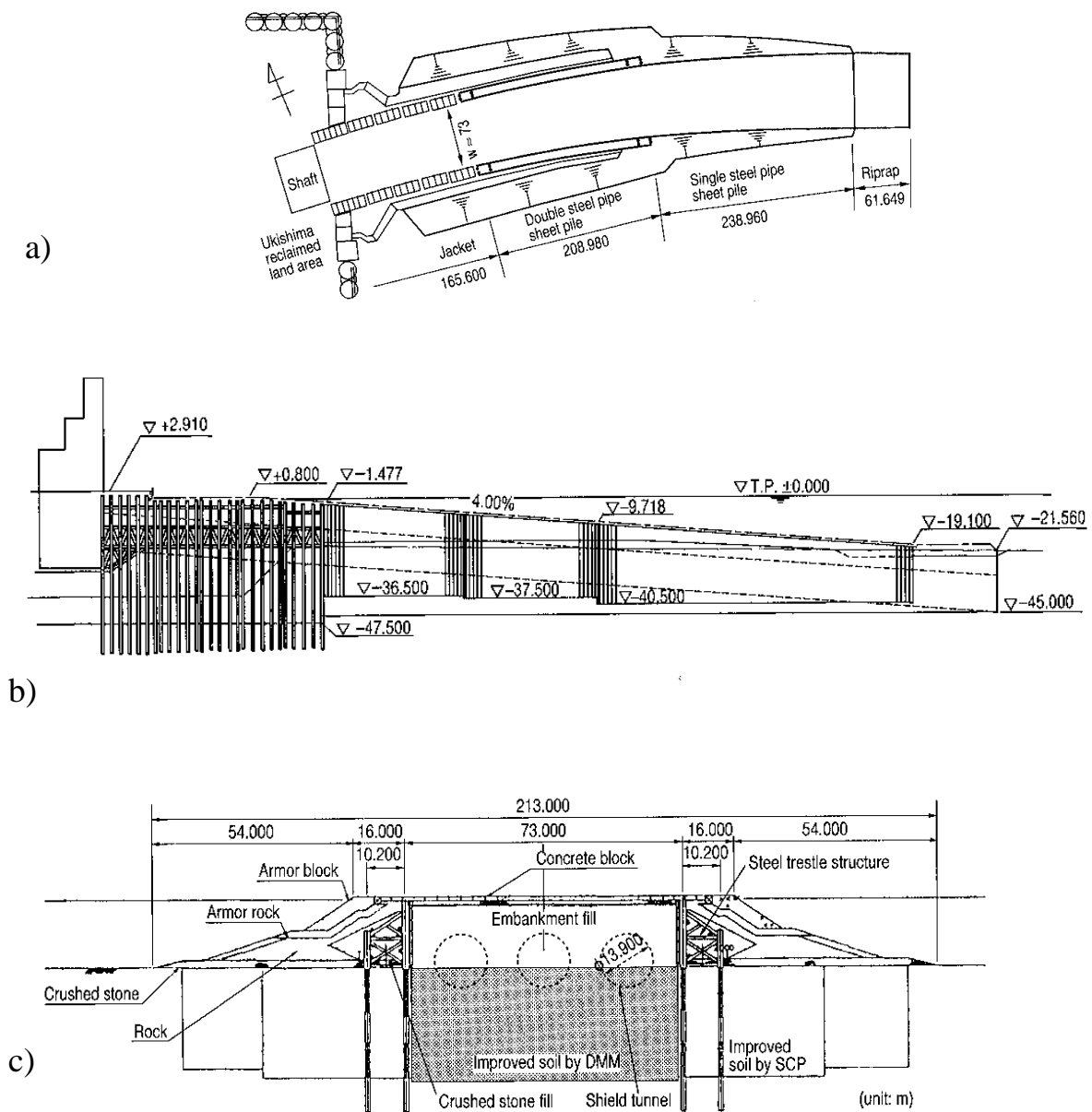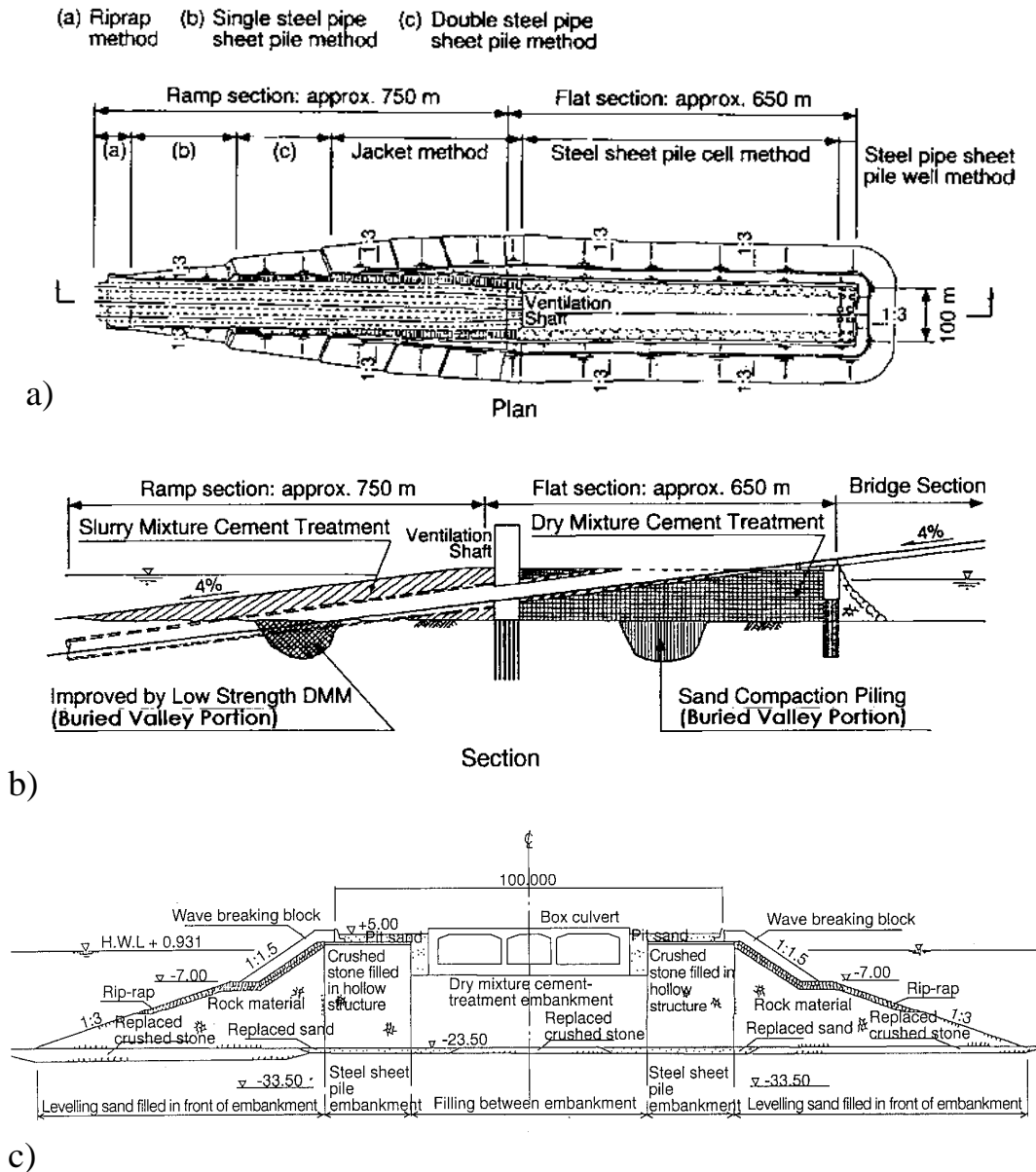

Figure 7: Ukishima Access.

The change in the stability analysis method for cement-mixed soil, described above, was one of the important technical steps that finally made it possible to adopt ground improvement techniques by cement-mixing in this project.

Slurry type cement-mixed sand for embankment fills

A new technique using a slurry of cement-mixed sand was adopted to construct the embankment fills at the ramp sections of the Ukishima Access (**Fig. 7** and **8**) and the Kisarazu man-made island (**Fig. 9**). The slurry type cement-mixed sand was also used to construct a cylindrical fill between the outer and inner steel jacket structures of the Kawasaki man-made island (Fig. 30, shown later). Based on the results from a comprehensive series of laboratory and field tests (Uchida et al., 1993; Tatsuoka et al., 1997), the slurry was designed to comprise sand 1,177 kg, seawater 520 kg, cement 80 kg and clay 110 kg for 1.0 m^3 of the slurry material (Table 1). This mixing proportion was determined to satisfy the following requirements:

Figure 8: Simplified cross-section of Ukishima Access (not to scale).

a) The total unit weight γ_t should be greater than 1.85 gf/cm^3 so that the weight of the fill overlying each tunnel can effectively resist the buoyant force of the tunnel. Therefore, sand was selected as the base material.

b) The average compressive strength of the material during the period of tunnelling work should be larger than 6 kgf/cm^2 (0.6 MPa) to ensure the stability of cutting face, while strength lower than about 30 kgf/cm^2 (3 MPa) was preferred for smooth tunnel excavation. As a compromise, the design average compressive strength at a curing period of 28 days was determined to be 10 kgf/cm^2 (1 MPa). The strength of the material was controlled mainly by the amount of cement.

c) Segregation of the components of the slurry during underwater placement should be as low as possible so that the properties of the placed material are close to those as designed and also seawater is not polluted. To this end, the bottom end of each tremmie pipe was controlled to be located immediately below the transient surface of the placed material so as to avoid free falling of slurry material in seawater. Furthermore, the hydraulic permeability of the slurry during placement work was reduced by adding an appropriate amount of clay, since a low permeability is essential to decrease the rate of segregation.

d) The hydraulic permeability of the solidified material during shield tunnelling work should be lower than 0.01 cm/sec so that the pressurised mud slurry does not penetrate into the fill made of this material at the cutting face. This was achieved by adding clay particles.

e) The slurry material should not be very viscous for smooth pressurised transportation in the pipes from an offshore slurry production plant to tremmie pipes, and also for smooth levelling off when being placed underwater. The viscosity was controlled mainly by adjusting the amount of water.

f) The cost of producing the material should be reasonably low. Chemical anti-segregation adhesive was not used because of its high cost, although it had been found that the use of a sufficient amount of this material be effective.

The slurry was produced on site in an offshore mixing plant installed in a 104 m-long vessel of 16,500 tonf (**Fig. 10**). The plant could produce 450 m^3 of the material per hour. The produced slurry was placed underwater using two sets of tremmie pipes, each set consisting of ten, installed in another vessel. As this system could place the material as fast as about 6,000 m^3 per day, producing one mound each half day, the construction of the fill embankments was accomplished in a relatively short duration. The fill embankments at the ramp sections of the Ukishima Access and the Kisarazu man-made island were constructed by using, respectively, 351,000 m^3 and 1,028,000 m^3 of this material for periods, respectively, from August to December 1993 and from July 1992 to February 1994.

The properties of the placed slurry mixture were examined and controlled during the placement work based on results from control tests that were periodically performed, including unconfined compression tests (U test) on specimens (5 cm in diameter; 10 cm high). The samples for the U tests were prepared by pouring slurry produced in the plant into empty moulds immediately before placement underwater. Soon after construction was completed, the properties of the material as placed were evaluated by retrieving undisturbed samples from the constructed fill embankments using the rotary core tube (RCT) sampling method (**Fig. 11**). A series of U tests and CU triaxial

Ramp section: approx. 750 m   Flat section: approx. 650 m

(a)   (b)   (c)   Jacket method   Steel sheet pile cell method   Steel pipe sheet pile well method

Ventilation Shaft

1:3

100 m

a)

Plan

Ramp section: approx. 750 m   Flat section: approx. 650 m   Bridge Section

Slurry Mixture Cement Treatment   Ventilation Shaft   Dry Mixture Cement Treatment   4%

4%

Improved by Low Strength DMM (Buried Valley Portion)

Sand Compaction Piling (Buried Valley Portion)

Section

b)

¢

100.000

Wave breaking block   Box culvert   Wave breaking block

H.W.L + 0.931   +5.00   Pit sand   Pit sand

Crushed stone filled in hollow structure   Crushed stone filled in hollow structure

-7.00   Rip-rap   Rock material   Dry mixture cement-treatment embankment   Rock material   -7.00   Rip-rap

1:3 Replaced crushed stone   Replaced sand   Replaced crushed stone   Replaced sand   Replaced crushed stone   1:3

-23.50

Steel sheet pile embankment   Steel sheet pile embankment

-33.50   -33.50

Levelling sand filled in front of embankment   Filling between embankment   Levelling sand filled in front of embankment

c)

Figure 9: Kisarazu man-made island.

compression (TC) tests were performed. In the CU TC tests, the samples were isotropically consolidated to the respective effective field overburden pressure σ'_v (in-situ). The compressive strengths that have been corrected to those at a curing time of 28 days are plotted in **Fig. 12**. The results showed that the density and strength values of the samples retrieved from the bore holes (denoted by the symbols of solid and hollow circles and solid and hollow squares) are satisfactory. Results from field down-hole seismic surveys performed in the bore holes (Fig. 11) showed that the fill had several relatively soft zones (i.e., relatively weak zones) (see Fig. 8 of Tatsuoka et al., 1997). It was considered, however, that the degree of homogeneity be acceptable.

Many of those strength values shown in Fig. 12 are noticeably higher than the specified design upper limit of compressive strength (i.e., 30 kgf/cm^2; 3 MPa). It is likely that this result is due to the following unexpected factors:
1) First, at the initial stage of this placement work, the strength of the material placed underwater was evaluated on site by the following method. Slurry material that flowed into small tubes (diameter about 10 cm) attached to the portion near the bottom end of a tremmie pipe in use for placement work was retrieved after the tremmie pipe was pulled up to above sea level. Then, U tests were performed on samples trimmed to a diameter of 5 cm. The strength was found to be much lower (by about 50 %) than those obtained from the control tests. Based on this observation, the amount of cement was increased from the original design value (80 kgf/m^3) to 100 kgf/m^3 (Table 1) so that the placed material would have a sufficiently large strength, but below the design upper limit value. The data points denoted by the symbols of asterisk and hollow triangle in Fig. 12 were obtained after the amount of

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cement was increased, from U tests using, respectively, samples obtained from small tubes attached to the portion near the bottom end of tremmie pipe and those obtained before placement work (i.e., control tests). It may be seen that the strength of the material retrieved from small tubes attached to tremmie pipes (symbol of asterisk) was only about 50 % of the respective value of the material before placement, obtained from the control tests (symbol of hollow triangle). The density of the underwater-placed material (symbol of asterisk) is also noticeably lower than the respective value of the material before underwater placement (symbol of hollow triangle). It is very likely therefore that the material retrieved from the small tubes had segregated when flowing into free water that was existing in the small tubes to a much larger extent than it had during the actual full-scale placement for the fill embankment. Therefore, it is very likely that the increase in the cement content is one reason for the unexpected high strength.

2) Secondly, it may be seen from Fig. 12 that the strength of the rotary core tube samples obtained from the constructed fill generally increases with depth, and they are larger than the respective value obtained from the control tests (symbol of hollow triangle). This fact can be explained, at least partly, by consolidation of the placed material due to its self-weight. The effect of accumulated heat by hydration of cement, which should be larger at larger depths, could be another cause. Underwater placement of this 'slurry' type cement-mixed sand to a total thickness of about 20 m for a relatively short duration was the first case, and, therefore, it is quite reasonable that such factors as described above could not be anticipated in advance. Despite this unexpected high strength of the solidified cement-mixed sand, any serious problems due to this high strength did not take place in the shield tunnelling work, as described later in this paper.
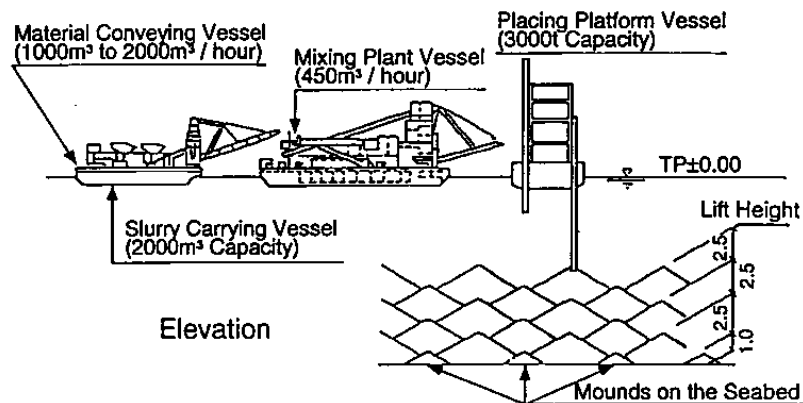
Figure 10: Underwater placement of a slurry of cement-mixed sand at the Ukishima Access and the Kisarazu man-made island.
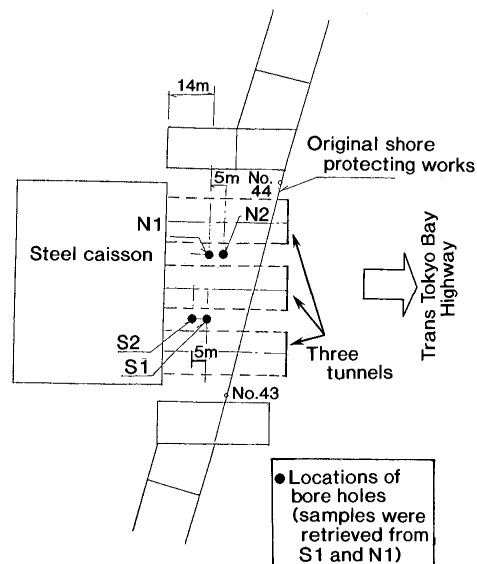
Figure 11: Location of bore holes in the embankment fill at Ukishima Access.

Low strength-type DMM to improve the existing soft clay deposits

Among the requirements for the fill embankment material, listed in the preceeding section, the requirements b), d) and f) were also important for the improvement method for the soft clay deposit that would support the fill while also retaining the shield tunnels at the ramp section at the Ukishima Access (**Fig. 13**). Despite that the size and depth of the soft clay deposit to be improved was much smaller, it was also the case with the ground improvement work at the Kawasaki man-made island (Fig. 30) and the Kisarazu man-made island (**Fig. 14**).

The ordinary type deep mixing method (DMM) has been used extensively in Japan for the purpose of improving existing soft clay deposits to considerable depths by mixing in-situ with cement slurry. In usual practice, the strength of treated clay is designed to be about 40 - 60 kgf/cm^2 (4 – 6 MPa) or more at a curing time of 28 days. At the design stage of this project, it was judged that the DMM would be suitable if the strength could be controlled so as to meet requirement b) (i.e., design average compressive strength of 10 kgf/cm^2; 1 MPa). After a full-scale field test (Uchida et al., 1993), the following mixing proportion was determined for the cement slurry; the cement content of 70 kgf per 1.0 m^3 of clay to be treated and a water/cement (W/C) ratio equal to 100 %, compared to 60 – 100 % in the ordinary DMM. This high W/C ratio was selected not to reduce the amount of cement slurry according to the reduction of
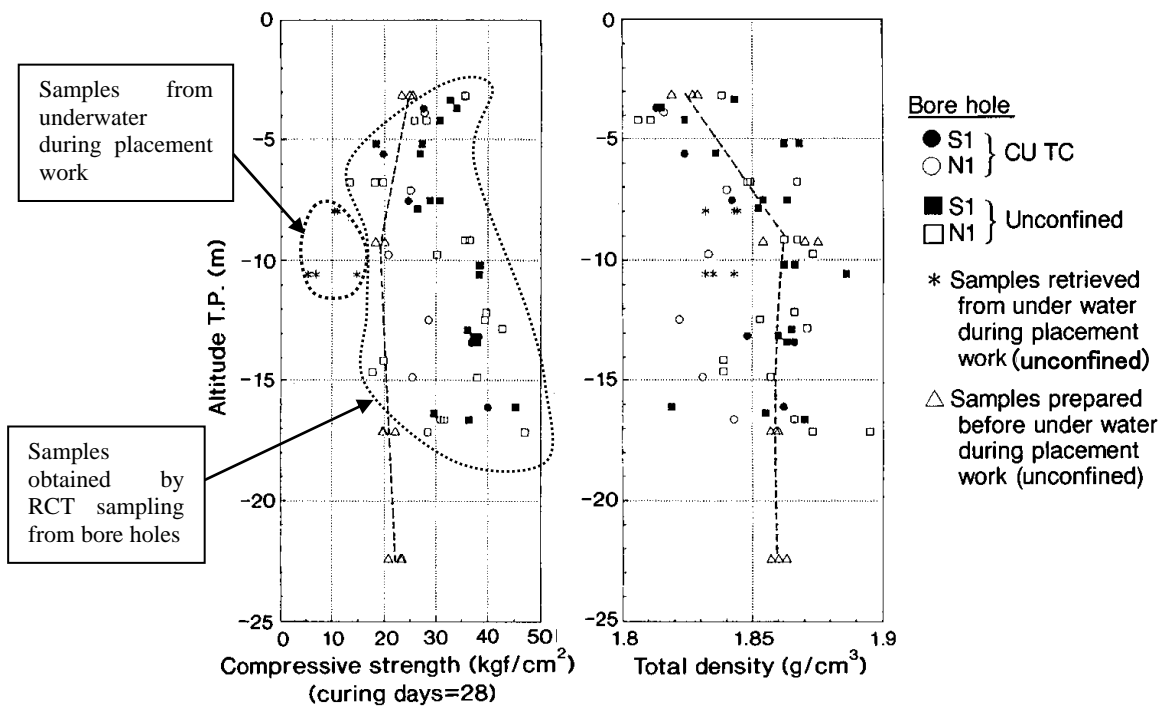
Figure 12:   Distribution of; a) compressive strength from U and CU TC tests; and b) total unit weight (saturated) of the samples for control tests, samples retrieved during placement, and RCT samples retrieved from the placed fill (see Fig. 11), Ukishima Access.

cement amount so that mixing with clay in-situ could be as homogeneous as possible.   This type of DMM has been called the low strength-type DMM.   The area ratio of the improved soil to the original untreated clay was designed to be as high as 99.44 %.   In total about 125,000 m^3 of soft Holocene clay was improved below the ramp section at the Ukishima Access.   For the ground improvement by the ordinary type DMM, a high W/C equal to 100 % was also used for the same reason as described above.

Fig. 15 shows a typical profile of the clay deposit at the Ukishima Access.   The original soft clay is a marine deposit having a plasticity index of about 35 – 53 and high natural water content w_n and low total unit weight γ_t as shown in Fig. 15.   It may be noted that the compressive strength q_{max} increased substantially by this ground improvement work.   The unconfined compressive strength q_u scatters relatively largely and some q_u values did not meet the requirement; i.e., the compressive strength equal to 6 - 30 kgf/cm^2 (0.6 – 3 MPa) with the average equal to 10 kgf/cm^2 (1 MPa).   On the other hand, the q_{max} values obtained from consolidated undrained (CU) triaxial compression (TC) tests with consolidation pressure σ'_c equal to σ'_v (in-situ) (symbol of cross mark) met the requirement.   It was considered that the q_u values were less reliable due to relatively large effects of sample disturbance (i.e., the field strength may be underestimated when based on the q_u values).   Down-hole seismic surveys were also performed, and the results showed that the fill embankment was reasonably homogeneous.

Dry mixture cement-treatment method for fill embankment

A new method called "the dry mixture cement-treatment method" (**Figs. 16a** and **17**) was developed and used to construct a fill in the space (volume 435,000 m^3) between the rows of steel-sheet pile cells on both sides of the flat part of the Kisarazu man-made island (see Fig. 9).   In this method, sand under natural, slightly wet conditions is mixed with cement in air, and anti-segregation liquid chemical adhesive is added immediately before dumping underwater.

At the initial design stage, it was planned to fill the space with ordinary sand and compact it by the sand compaction pile method.   However, the results from a series of cyclic undrained torsional shear tests (Tatsuoka et al., 1982) showed that it would be necessary to compact the sand fill to a relative density of about 85 % or more to prevent liquefaction during earthquakes.   This method was abandoned, since it could not be confirmed that the sand fill could be compacted to that degree of density.   At the same time, the construction period was required to be as short as possible so that the ground surface of the flat part could be used as soon as possible after its completion as a yard for subsequent construction work.   For these two reasons, the dry mixture cement-treatment method was considered to be suitable, despite its relatively high cost.

This ground improvement method was required to satisfy the following requirements:

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a) the fill ca

a) A–A section

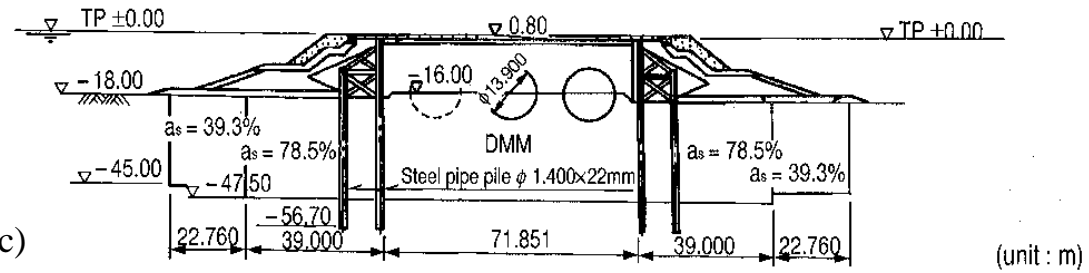$a_{sw} = 25.9\%$
$a_{s1} = 78.5\%$
675.189
$a_{sw} = 25.9\%$
$a_{s1} = 39.3\%$
$a_s = 39.3\%$
$a_s = 25.9\%$
$a_s = 19.6\%$
$a_s = 9.8\%$
Shaft
$a_s = 78.5\%$
$a_s = 58.9\%$
A $a_s = 30.1\%$
$a_{sw} = 25.9\%$
$a_{s1} = 78.5\%$
DMM
$a_s = 25.9\%$
$a_s = 78.5\%$
$a_s = 58.9\%$
$a_{sw} = 25.9\%$
$a_{s1} = 78.5\%$
$a_{sw} = 25.9\%$
$a_s = 39.3\%$
$a_s = 19.6\%$
$a_s = 9.8\%$
$a_s = 39.3\%$
$a_{s1} = 39.3\%$
A
B
B

b) B–B section

165.600   Jacket
208.980   Double steel pipe sheet pile
238.960   Single steel pipe sheet pile
61.649   Riprap
+2.91
−18.30
4%
−36.09
−39.00   Ac1
$a_s = 25.9\%$
Existing sand replacement
−45.00   Ac2
$a_{s1} = 39.3\%$
−47.00   Plhc
$a_s = 78.5\%$
$a_s = 58.9\%$
(Shield tunnel)
−51.00   Plhs
$a_s = 30.1\%$
−53.20   Plhc

Ac1 : Alluvial clayey soil (upper)
Ac2 : Alluvial clayey soil (lower)
Plhc : Pleistocene clayey soil
Plhs : Pleistocene sandy soil

c)

▽ TP ±0.00   ▽ 0.80   ▽ TP +0.00
▽ −18.00   −16.00 $\phi 13.900$
$a_s = 39.3\%$
$a_s = 78.5\%$ DMM $a_s = 78.5\%$
−45.00   Steel pipe pile ϕ 1.400×22mm $a_s = 39.3\%$
−47.50
−56.70
22.760   39.000   71.851   39.000   22.760
(unit : m)

Figure 13:   Ground improvement by the sand compaction pile method and the low strength-type DMM at Ukishima Access.

b) the fill is very stable and will not exhibit noticeable settlement during earthquakes; and

c) reasonably small earth pressure works on the steel sheet pile cell structures.

To meet these requirements, the design compressive strength was determined to be 4 kgf/cm^2 (0.4 MPa) or more. This value is significantly lower than that required for the slurry type cement-mixed sand used for the construction of the fill embankments, retaining the shield tunnels, at the ramp sections of the Ukishima Access and Kisarazu man-made island.

Based on results from large-scale placement tests performed in advance, the mixing proportion of the material was determined, which is, for a mixture of 1 m^3, 1,330 kg of dry weight of Sengenyama sand having a natural water content, 100 kg of cement and 110 g of anti-segregation chemical adhesive. Samples obtained from the large-scale placement tests had an average value of q_u = 6.3 kgf/cm^2 (0.62 MPa) at a curing time of 91 days and a dry unit weight of 1.80 - 1.85 gf/cm^3 and a hydraulic permeability of 10^{-3} to 10^{-4} cm/sec (Tatsuoka et al., 1997).

With this method, the quality of the placed material depends largely on details of the placement method. A special double-tube chute was developed so as to minimise the degree of segregation (**Fig. 16b**). That is, the bottom end of the outer tube (or pipe) is in contact with the transient surface of placed material and the water is circulated as much as possible inside the chute. The seawater is pushed out by the material dumped from the bottom end of the internal tube, moves up in the space between the outer and internal tubes and then is sucked into the top part of the internal tube. By this method, the spreading of the material when dumped underwater could be effectively restrained. This mixing and placement method exhibited a much faster construction speed and a lower cost when compared with the slurry method using tremmie pipes.
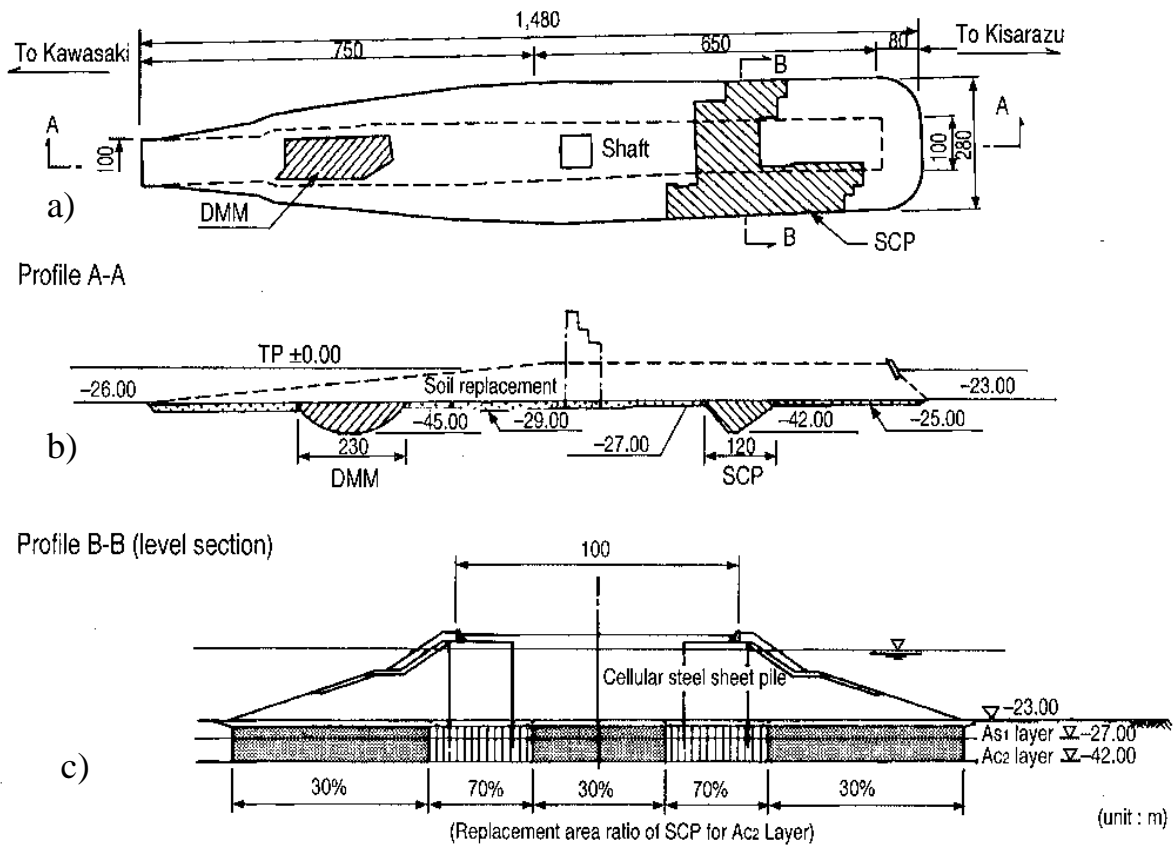
Figure 14: Ground improvement by the sand compaction pile method and the low strength-type DMM at Kisarazu man-made island.
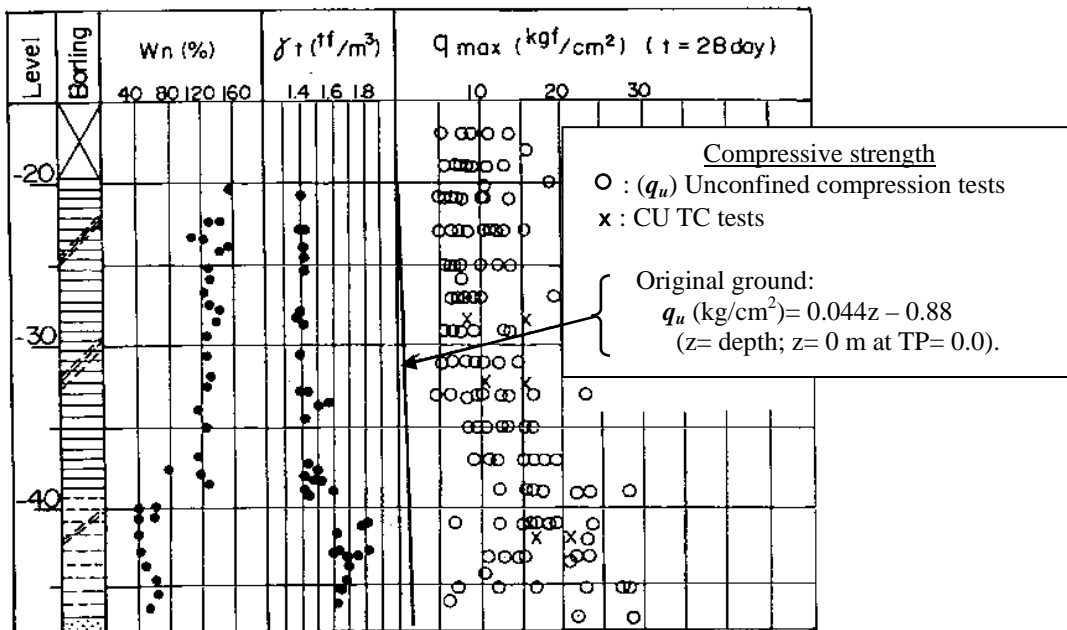
Figure 15: A typical soil profile of the clay deposit improved by the low strength-type DMM, Ukishima Access.

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a)
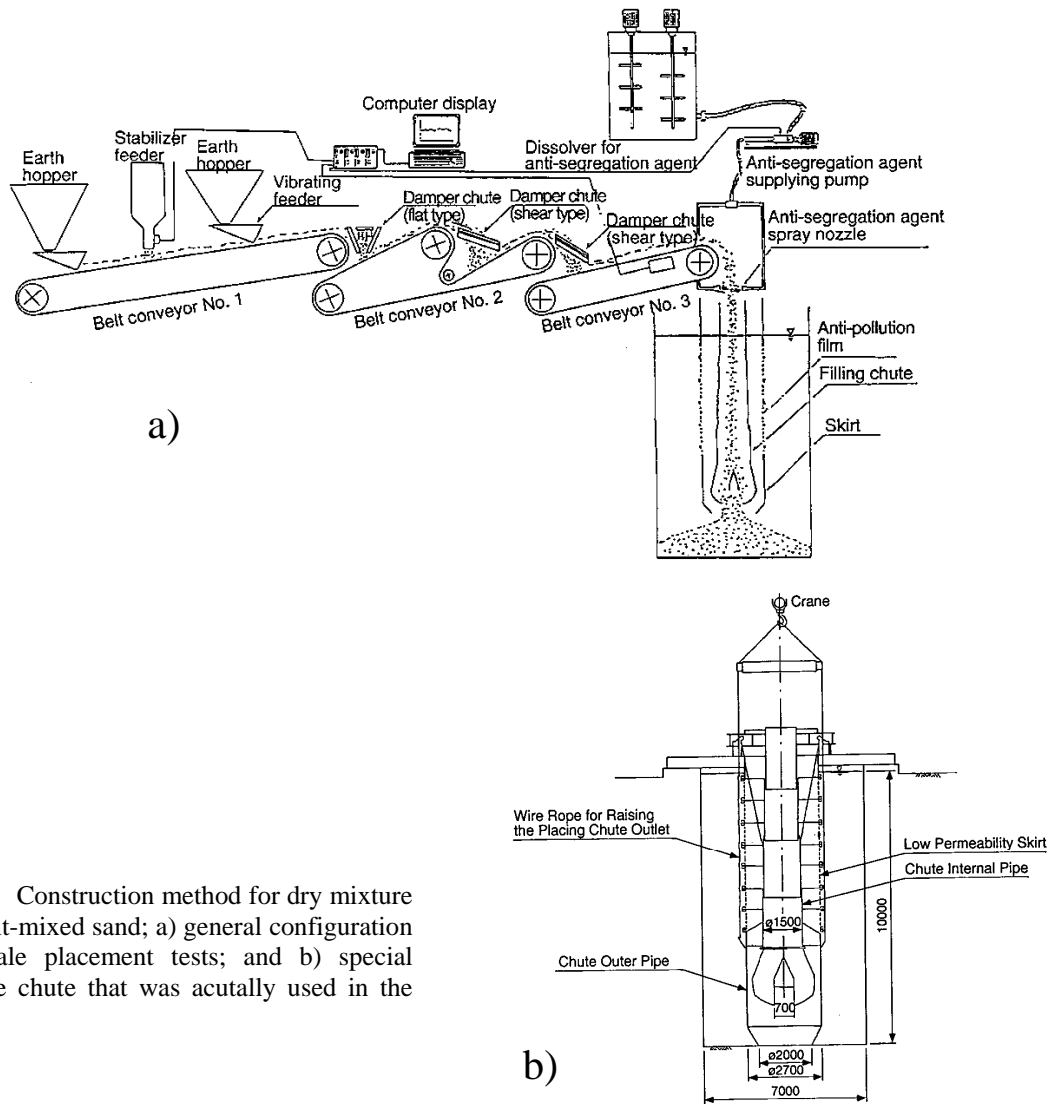
Figure 16: Construction method for dry mixture type cement-mixed sand; a) general configuration for full-scale placement tests; and b) special double-tube chute that was acutally used in the project.

b)
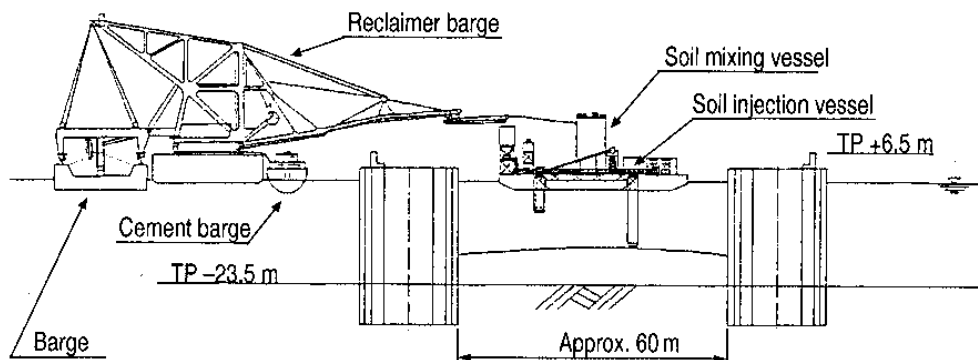
Figure 17: Placement work of the dry mixture type cement-mixed sand, Kisarazu man-made island.

Results from a series of field investigations, including undisturbed sampling, U tests, CU TC tests and field seismic surveys that were performed in the constructed fill embankment, showed that the properties of the placed material satisfy the prescribed requirements (Tatsuoka et al., 1997).

SHIELD TUNNELS

The tunnels were constructed by means of blind type shield machines using pressurised mud slurry for the stabilisation of cutting face and smooth transportation of excavated soil to an onshore treatment plant (**Fig. 18**). The

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typical arrangement of the shield tunnels is presented in **Fig. 19a**. The centre-to-centre separation between the tunnels was set at about 2D (D= the tunnel external diameter equal to 13.9 m) in the seabed section and about 1.5D in the ramp sections. Eight shield tunnelling machines progressed simultaneously from both ends of each of the four tunnels; a pair from the Ukishima Access, two pairs from the Kawasaki man-made island in opposite directions, and the final pair from the Kisarazu man-made island (Fig. 19a). Shield machines started proceeding into the ground from a large steel
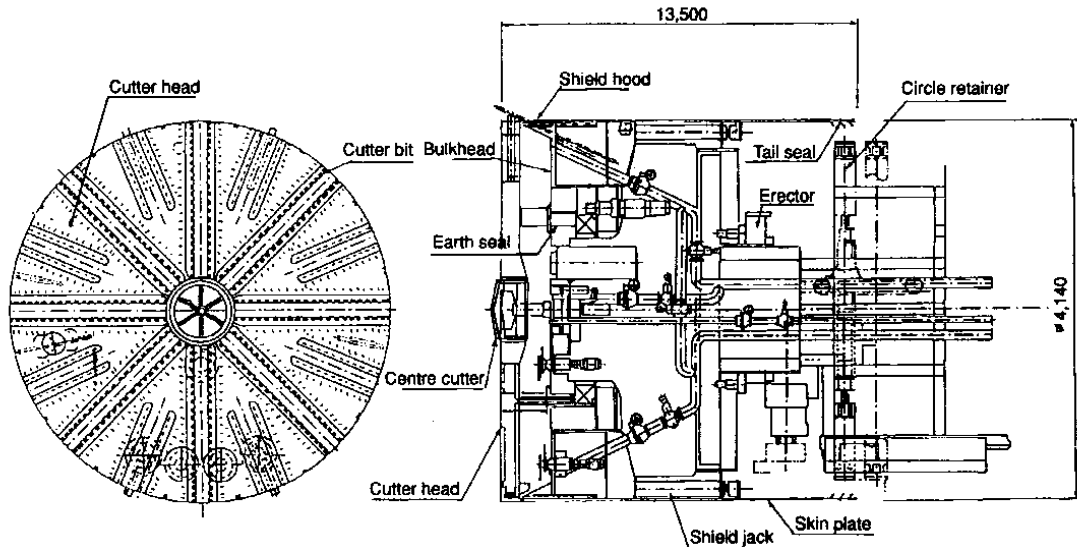
Figure 18: Schematic diagram showing the pressurised mud slurry shield tunnel boring machine.

a)
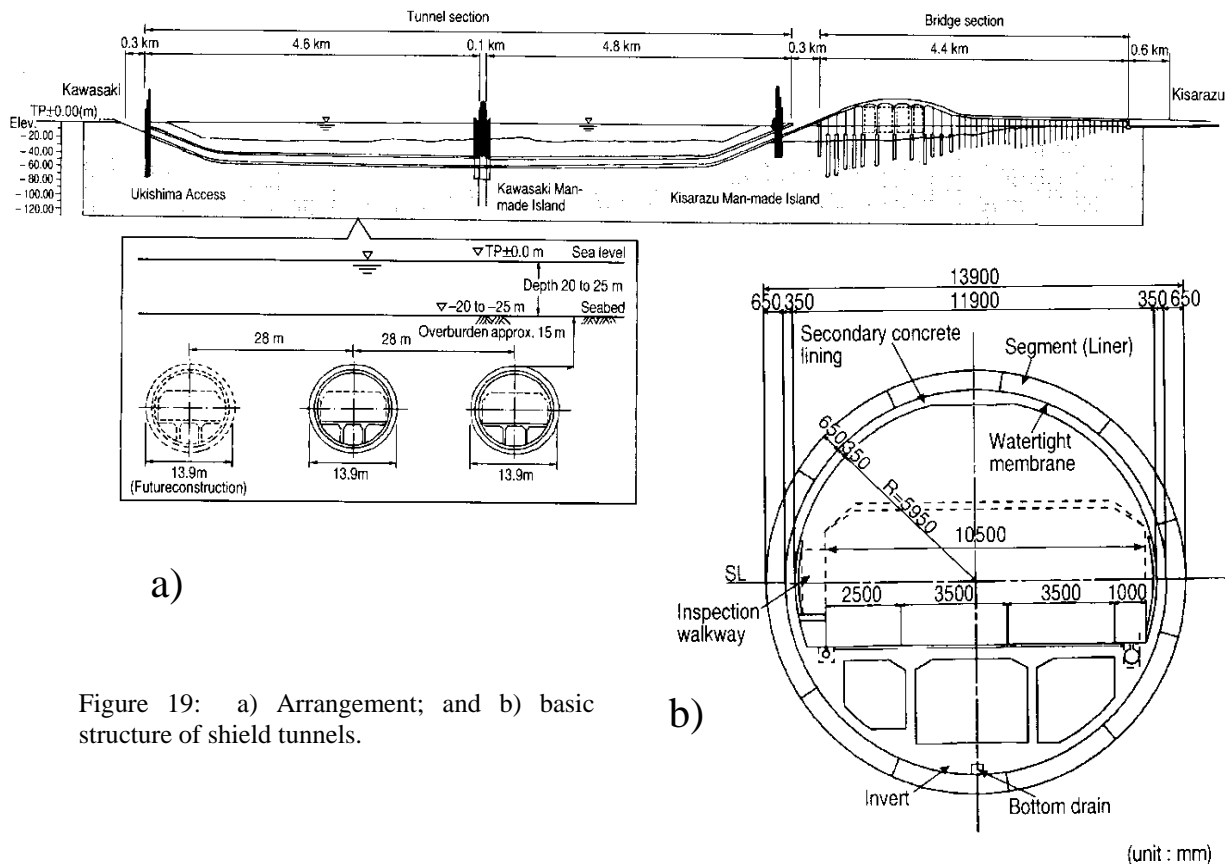
Figure 19: a) Arrangement; and b) basic structure of shield tunnels.

b)

(unit : mm)

caisson at the Ukishima Access (Figs. 7 and 8) and the Kisarazu man-made island (Fig. 9), which became the ventilation towers after the highway was completed, and from a diaphragm wall at the Kawasaki man-made island (Fig. 30).

For safe and smooth shield tunnelling work, to avoid seawater pollution and for the permanent stability of the

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completed structures, it was decided;

1) to construct the fill embankments in which the tunnels were to be constructed by using cement-mixed sand; and
2) to improve by in-situ cement-mixing the existing soft clay deposits which were to support the fill embankments and through which the tunnels were to be constructed.

The construction of the fill embankments at the ramp sections of the Ukishima Access and the Kisarazu man-made island was inevitable for the safe construction of the shield tunnels through beneath the sea bottom. This is because the shield tunnel construction should start from a dry area, due to the fact that starting shield tunnelling construction underwater is technically extremely difficult.

Some design issues

The basic structure of the tunnel, which is shown in **Fig. 19b**, was determined based on a number of design considerations, as follows (Uchida et al., 1993):

1) The primary lining was designed to resist the external load ensuring a high water-tightness and durability:
2) It was decided to construct a secondary lining, which serves as additional mass against uplift due to the buoyant force of the tunnel and can protect the primary lining in case of traffic accident including fire.
3) A water-proof membrane was installed between the primary and secondary linings and the connection between the adjacent segments of the primary lining was made water-tight.
4) The structure of the tunnel itself and the connections between the tunnel, the ventilation tower and the diaphragm wall were made flexible to absorb possible large deformations during an earthquake.

Successive excavation in stiff and very soft deposits

The technical questions associated with a long and continuous shield tunnel drive in deposits having extremely different strengths included the following:

1) whether efficient (or smooth) excavation could be achieved in; a) the fills of cement-mixed sand at the ramp sections at the Ukishima Access and the Kisarazu man-made island; and b) the existing soft clay deposit that had been improved by the low strength-type DMM below the ramp section at the Ukishima Access and adjacent to the diaphragm wall of the Kawasaki man-made island;
2) whether safe tunnelling work in the existing very soft Holocene clay deposit at a very shallow depth, in front of the ramp section at the Ukishima Access, could be achieved;
3) whether several mechanical parts of shield tunnelling machine had sufficient wearing resistance for a long tunnel drive; and
4) whether safe underground connection at the cutting faces of two shield tunnel could be made.

Excavation in the cement-mixed soil

Fig. 20 shows the distribution with depth of the compressive strengths, converted to those at a curing time of 28 days, in the fill of cement-mixed sand and the soft clay deposit improved by the low strength-type DMM, obtained from a series of unconfined and CU triaxial compression tests on undisturbed samples retrieved from the Ukishima Access (Ouchi et al., 1997). For the cement-mixed soft clay deposit, due to the effects of sample disturbance associated with rotary core tube sampling, the strength from CUTC tests on
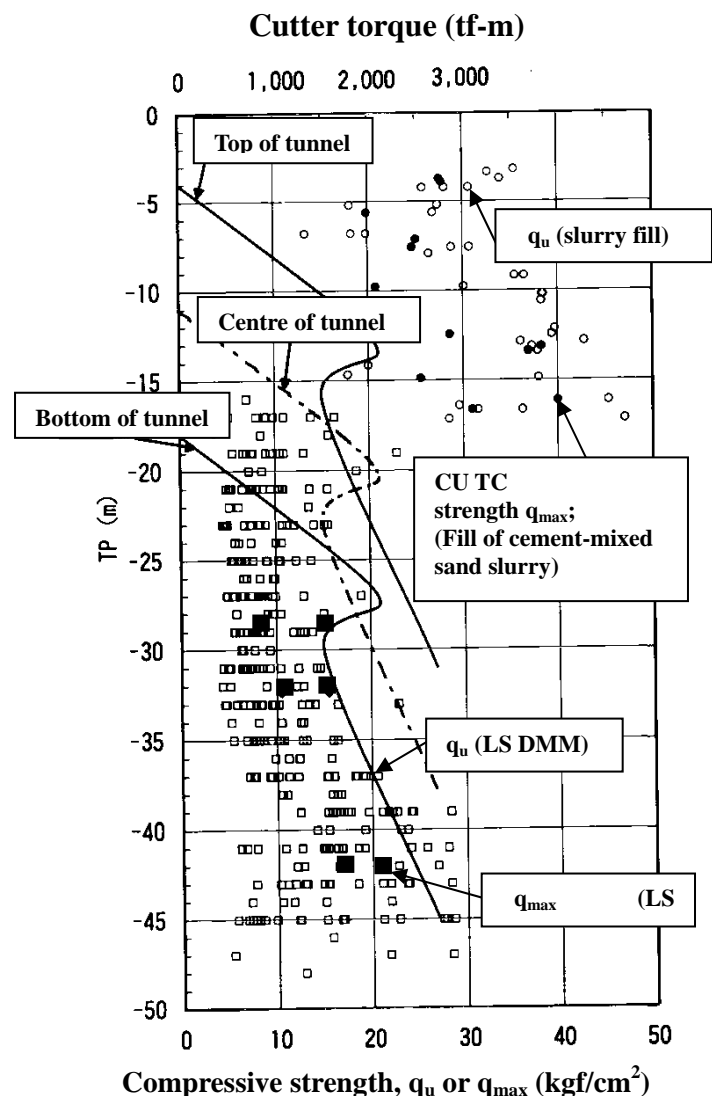

Figure 20: Distribution of compressive strength of cement-mixed sand and the soft clay deposit improved by the low strength-type DMM and the relationship between the depth of the cutter head of shield and the cutting torque, Ukishima Access (Ouchi et al., 1997).

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specimens consolidated to the field pressure level is much more reliable than the unconfined compressive strength. As stated before, the strength of the fills made of cement-mixed sand and the existing soft clay deposit improved by the low strength-type DMM was designed to be as small as possible for smooth tunnelling work while being strong and rigid enough for the stability and small deformation of the fill at the ramp section. However, the strength of the cement-mixed sand fill had become larger than the design value for several unexpected reasons (Fig. 12). Despite the above, the actual tunnelling work in the cement-mixed soil deposit was smooth with an average proceeding speed of the shield tunnelling machine of 25 – 30 mm/min with the maximum speed of about 40 mm/min in excavation of the cement-mixed soil (the fill and the existing clay deposit). A similar tunnelling speed was recorded during the work in the unimproved very soft clay deposit.

The major factors controlling the cutting torque acting at the cutter head of the shield machine are the shear resistance of the ground at the cutting face and the frictional resistance along the interface between the cutter head and the ground. **Figs. 21** and **22** show the relationships between the recorded cutting torque (CT) (tonf-m) at the cutter head of shield machine and the excavation distance for the two tunnels at the ramp section and in the ground in front of the ramp section at the Ukishima Access and the Kisarazu man-made island, respectively. The total distance shown in the figures is 1.5 km. The soil types indicated in the top of each figure are those that were encountered at the cutter head of shield machine. The following trends of behaviour may be seen:

Slurry fill & LS DMM	Low strength-type DMM	Holo-cene (Ac1) soil layer	Pleistocene (Dc) soil layer
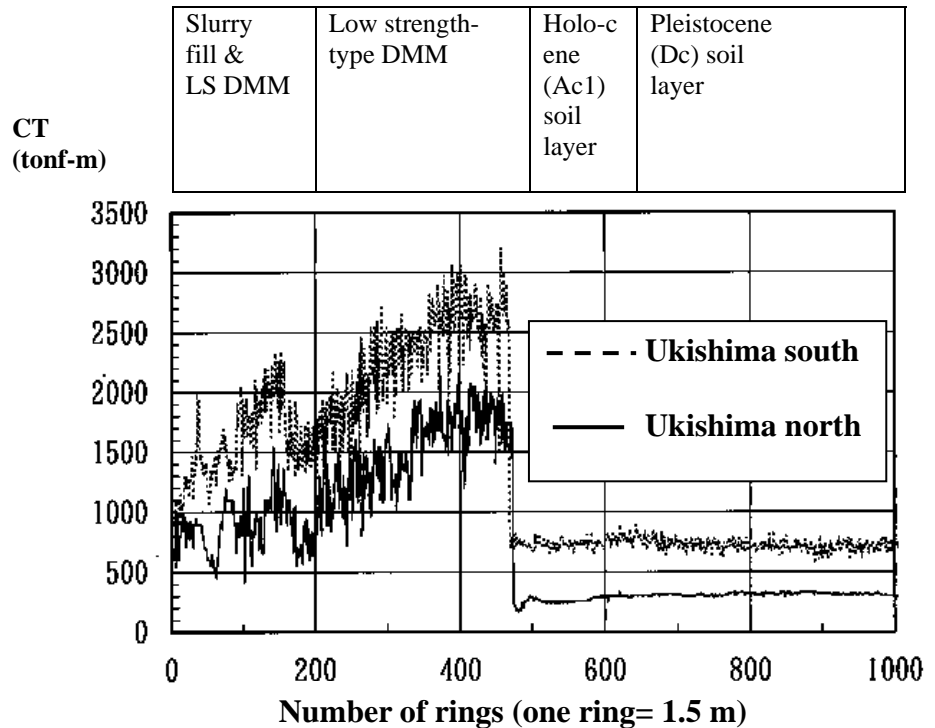
Figure 21: Relationships between cutting torque and excavation distance for two shield tunnels, Ukishima Access (Ouchi et al., 19987).

Slurry fill	Slurry fill & LS DMM	LS DMM & Pleisto-cene soil	Pleistocene (Dc) soil layer
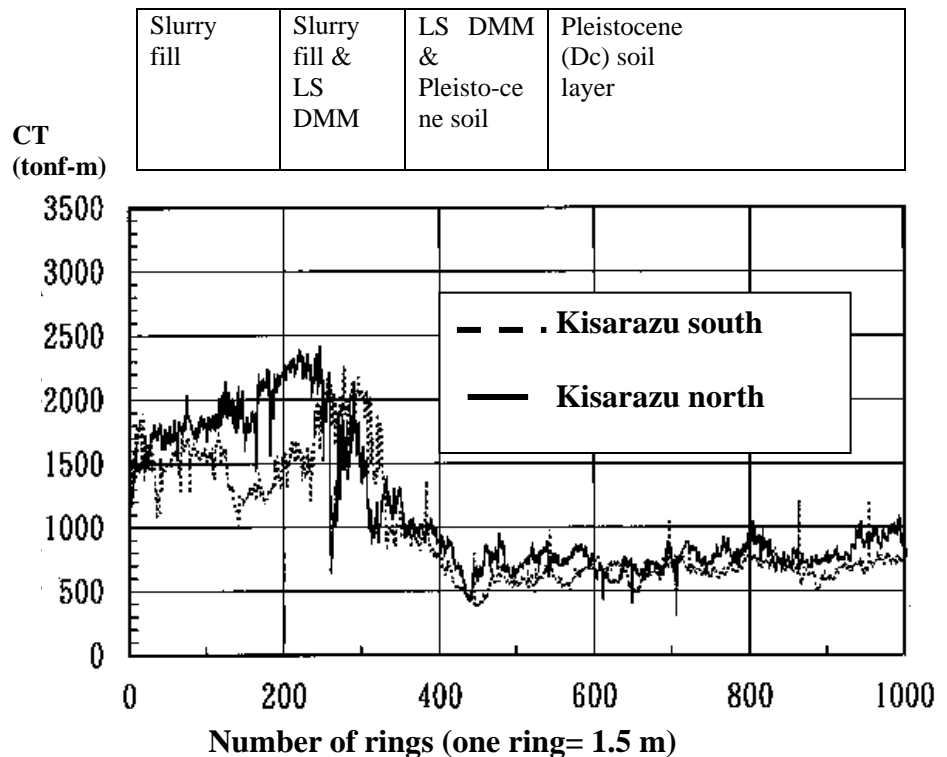
Figure 22: Relationships between cutting torque and excavation distance for two shield tunnels, Kisarazu man-made island (Ouchi et al., 1997).

1) The measured cutting torque when excavating in the cement-mixed soil was considerably higher than that when excavating in the existing un-improved soil deposits.

2) At the Ukishima access (Fig. 21), the measured cutting torque first increased with the increase in the depth of the cutter head in the cement-mixed sand fill. Then, the cutting torque temporarily decreased as the cutter head started entering the soft clay deposit improved by the low strength-type DMM (see Fig. 20). As the location of the cutter head of the shield machine became deeper in the soft clay improved by the low strength-type DMM, the cutting torque started increasing again. This would be due to the fact that both shear strength of cement-mixed clay and frictional resistance at the interface between the cutter head of shield tunnel and the cement-mixed clay increased with the increase in the depth.
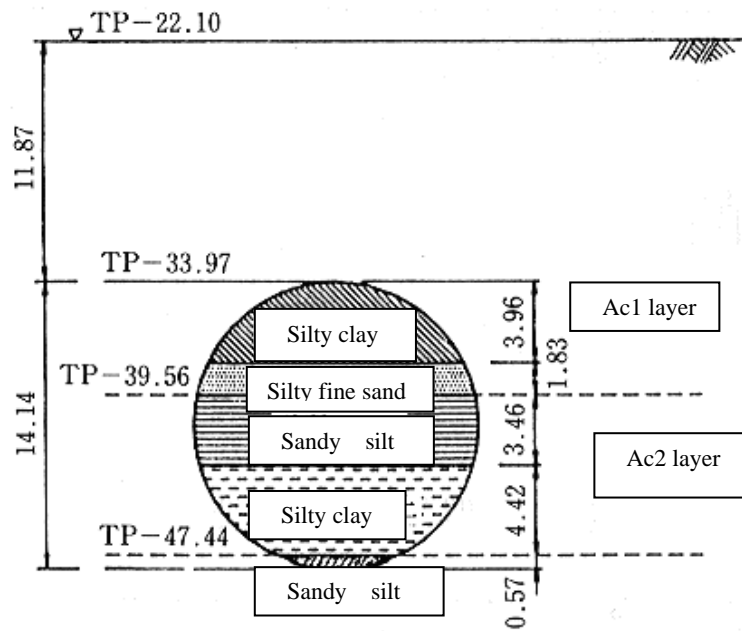

Figure 23: Typical soil conditions at the cutting face when proceeding in the un-improved soft clay deposit in front of the ramp section, Ukishima Access (Wakatsuki and Okazaki, 1998).

3) At the Kisarazu man-made island (Fig. 22), the increase in the cutting torque with the increase in the depth of the cutter head in the existing ground, which had been partly improved by the low strength-type DMM (Fig. 14), was not as significant as it was at the Ukishima Access. It was due to a much smaller thickness of the improved soft clay deposit at the Kisarazu man-made island (Fig. 14).

It has been known that the cutting torque is generally proportional to D^3, where D is the diameter of the cutter head, under otherwise the same conditions. In one of the recent on-shore shield tunnelling projects, a shield machine with an external diameter of 4.68 m proceeded in a sedimentary soft rock deposit, having strengths similar to those of the cement-mixed sand and clay shown in Fig. 20. The cutting torque was 50 – 80 tonf-m (Ouchi et al., 1997). This value is converted to 1,400 – 2,200 tonf-m for a shield diameter of 14.14 m, which is consistent with those measured at the Ukishima site (Fig. 21).

Excavation in unimproved soft clay at a shallow depth (Wakatsuki and Okazaki, 1998)

It may be seen from Fig. 21 that the cutting torque suddenly dropped to a very small value as the cutter head of the shield machine entered into an unimproved very soft clay deposit in front of the ramp section of the Ukishima Access. **Fig. 23** shows a typical soil condition that was encountered at the cutting face. The following are the two major potential serious problems with this tunnelling work in such a very soft clay deposit at a very shallow depth;

1) active and passive failure in the soft clay ground in front of the cutter head of shield machine; and

2) floatation of the tunnel due to the buoyant force of the tunnel due to too small weight and too low shear resistance of the overlying thin soil deposit.

To overcome the first problem, it was necessary to extremely carefully control the mud slurry pressure. That is, the mud pressure at the top level of the cutter head of the shield machine should be controlled in such that:

1) the mud slurry pressure does not exceed the total overburden pressure in the ground at the cutting face so that the mud slurry does not penetrate into the soft clay deposit; while

2) the mud slurry pressure does not become smaller than the active earth pressure (in terms of total stress) at the cutting face so that the soft clay deposit does not exhibit active failure towards the shield tunnel.

The range of the two limit pressures was only 0.404 kgf/cm^2 (39.6 kPa), which was very close to the range of possible and inevitable fluctuation in the mud slurry pressure, which was estimated to be equal to about 0.4 kgf/cm^2 (about 40 kPa). This extreme difficult control was accomplished with monitoring the pore water pressure in the clay deposit immediately below the seabed and displacements at the seabed above the shield tunnel.

To overcome the second problem, as a conservative decision, the resistance of the overlying soil deposit against floatation of the shield tunnel was estimated to be equal to the total weight of the soil mass that is overlying the plan

area of the shield tunnel, ignoring the shear resistance of soil along the vertical planes above the ends of the tunnel diameter (**Fig. 24**).   Then, the sum of the total weight of the tunnel (before installing the secondary inner lining) and the overlying soil deposit was smaller than the tunnel buoyant force by 22.7 tonf/m. To alleviate this problem, the following temporary weight was installed inside the tunnel; a) heavy weight steel I-beams, used as sleepers of the transportation railway; b) additional rails, placed between each adjacent sleepers; and c) steel ingots, placed in the carriages.   In addition, part of the weight of the shield machine was applied to the segments immediately behind the machine to prevent the floatation of the latter.   The upward displacement of the RC segments of the tunnel was continuously monitored. The upward displacement of the tunnel was found to become largest behind the tail of the shield machine, which was,

Figure 24:   Schematic diagram showing the balance between the weight of tunnel and soil and the buoyant force acting to the tunnel, Ukishima Access (Wakatsuki and Okazaki, 1998).

however, less than 10 mm, not indicating a danger of floatation of the tunnel.   At a distance of 100 m or more behind the shield tunnelling machine, the upward displacement of the tunnel became practically zero.

Underground connection of shield tunnelling machines
There were the following potential problems with such a long tunnel drive as 2.0 – 2.5 km in this project:
1) The total construction period of the highway project is controlled by the construction period of the shield tunnels, which depends on the largest drive length of shield tunnelling.
2) The wearing resistance in the tail seal, earth seal and cutting bits of shield machine may become a problem when the tunnel drive length becomes too large.
3) With the increase in the drive length, the equipment for supply and recovery of mud slurry between the shield machine and the onshore plant becomes longer.
The problems associated with a long tunnel drive were reduced by making an underground connection in the centre of each tunnel; i.e., each pair of machines was connected to each other at four locations (Matsuoka et al., 1998) (**Fig. 25**).   To this end, the ground around the connection section was frozen in advance before connecting the cutter
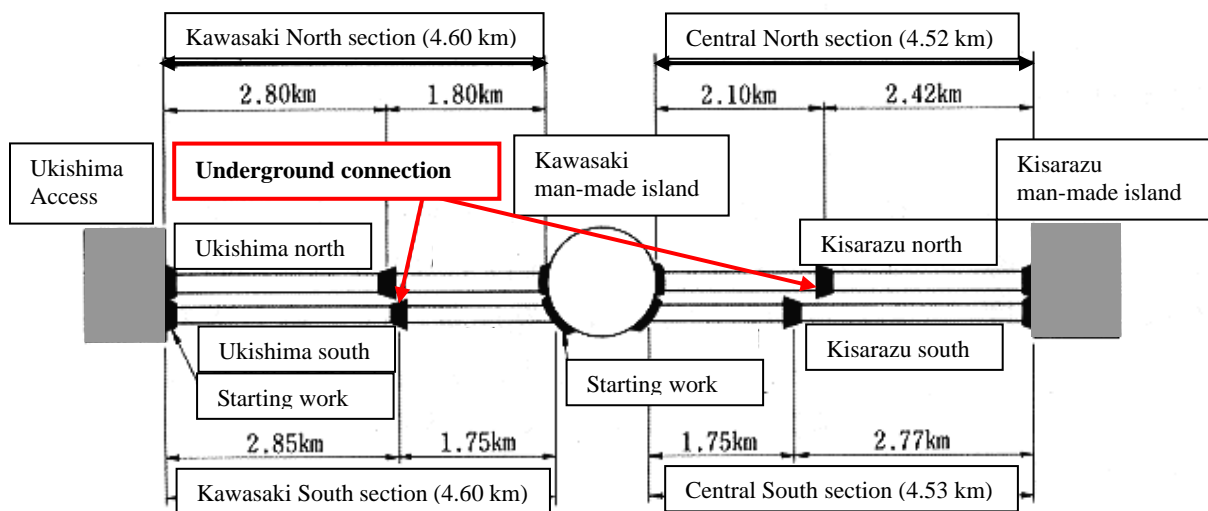

Figure 25:   Locations of underground connection shield tunnel machines (Matsuoka et al., 1998).

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heads of two shield machines as follows.   To reduce the size of frozen soil (therefore the total time and cost) for the tunnel connection work, the following measures were taken (**Fig. 26**):

1) The freezing pipes were extruded only from the shield hood of the machine that arrived first at the connection point. All the other equipment for the ground freezing work had been equipped in advance to each shield machine.   The freezing pipes attached along the outside face of the shield hood were used for the ground freezing work.

2) The distance between the cutter heads of the two shield machines was made as small as possible before the start of ground freezing work by bringing back the central cutter, located at the centre of the cutter head, towards the inside
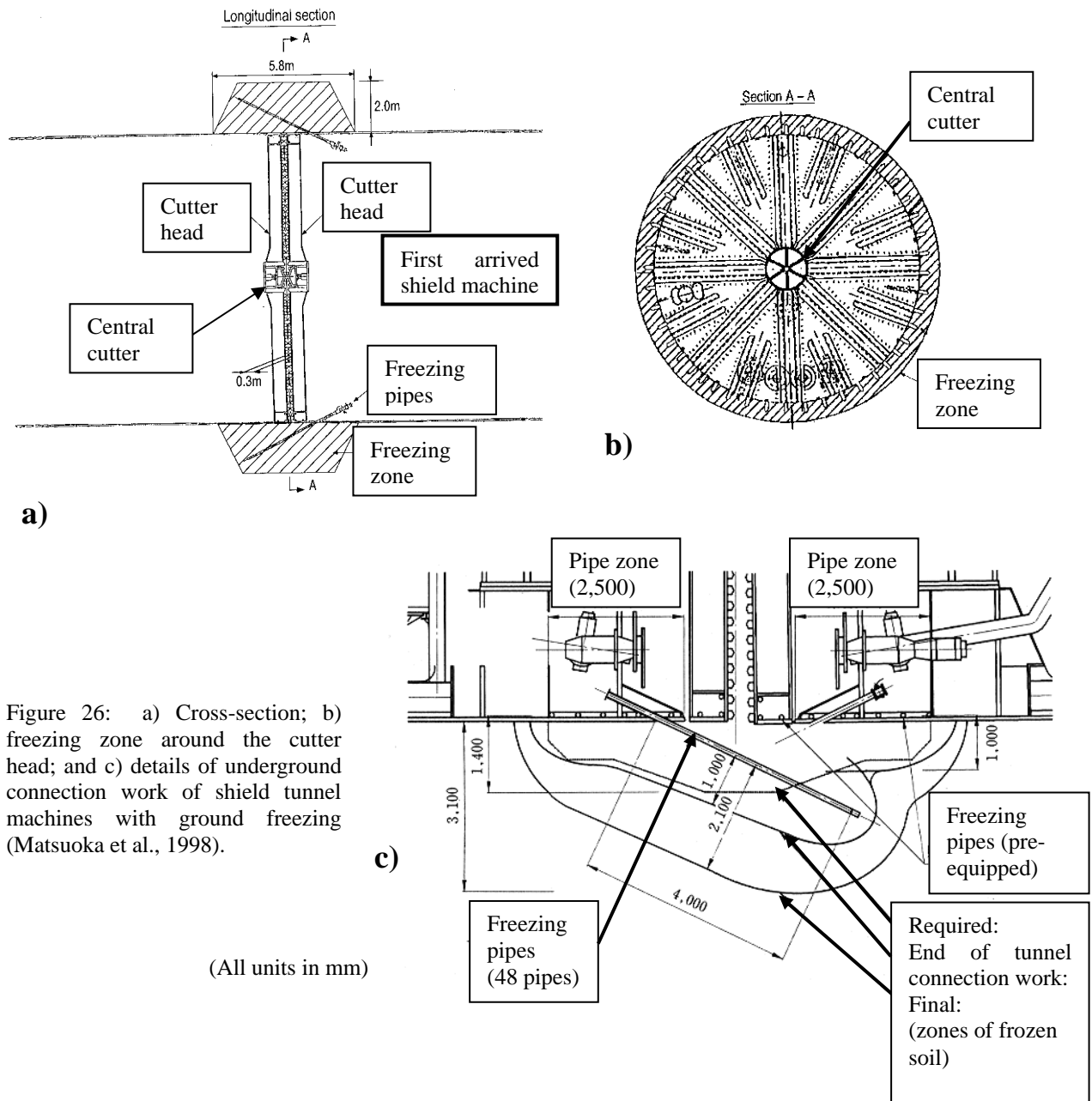
Figure 26:   a) Cross-section; b) freezing zone around the cutter head; and c) details of underground connection work of shield tunnel machines with ground freezing (Matsuoka et al., 1998).

(All units in mm)

of the shield machine (Fig. 26).   The locations of two cutter heads were carefully controlled and adjusted by means of ground investigation by boring from the shield machine.

3) The shield was not removed before the connection work (and left forever) to safeguard the tunnel against earth and water pressures.

The target allowable maximum errors were a distance of 50 mm in the direction perpendicular to the tunnel axis, an angle of 0.8 degrees between the two tunnel axes and a distance between the two cutter head of 0.5 m.   These values could be satisfied.

   Fig. 27 shows the time histories of temperature in the frozen soil zones immediately outside the shield around the connection section for the south tunnel between the Ukishima Access and the Kawasaki man-made island.   The connection section was located in a Pleistocene sandy deposit, including thin clay layers (D1s layer in Fig. 2).   The
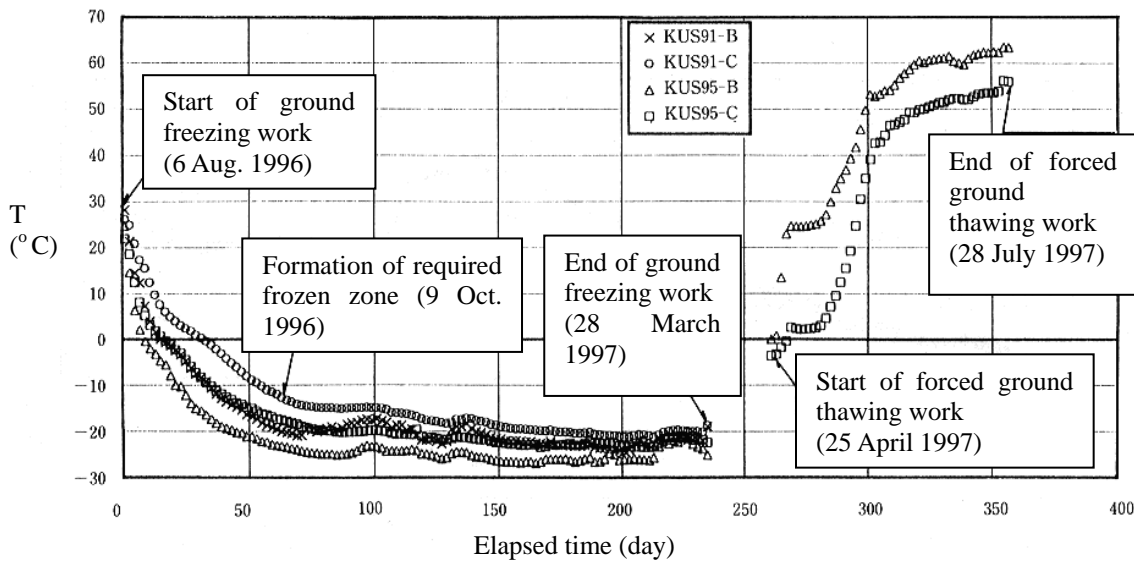
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Figure 27: Time histories of temperature around the connection point between Ukishima Access and Kawasaki m-m island; points KUS91 & 95 are above and below the tunnel diameter (Matsuoka et al., 1998).
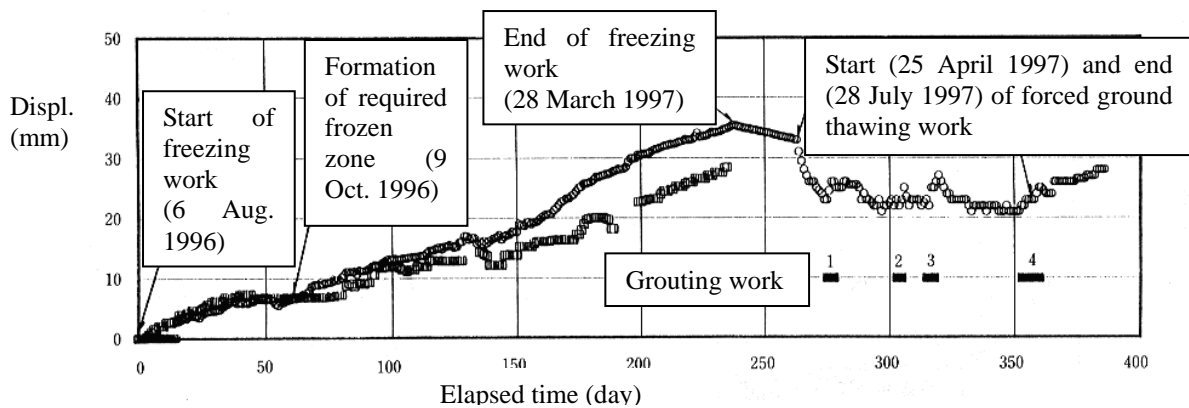

Figure 28: Time histories of vertical displacement of shield tunnel machines at the connection section between Ukishima Access and Kawasaki m-m island (Matsuoka et al., 1998).

total period to complete the ground freezing work was 64 days, which was longer than the planned period (56 days). This was due to the fact that the initial ground temperature (25° – 30° C) was higher than the predicted value (18° C) and the heat flow from the shield machine and the water content in the frozen soil zone were larger than predicted.

Fig. 28 shows the time histories of vertical displacement of the two shield machines at the connection section during the connection work, while **Fig. 29** shows the vertical locations of the tunnel axial of the two shield machines when the planned frozen soil zone was formed and when the freezing operation was ceased after having connected the shield machines. The maximum heaving was 28.5 mm and 35.4 mm for the two tunnels from the Ukishima Access and from the Kawasaki man-made island, which was about 25 % of the predicted value. To avoid residual deformation of the frozen ground upon natural slow thawing for a long duration to occur in an uncontrollable manner, the frozen ground was forced to thaw for a short period by supplying hot water in the freezing pipes. Voids in the ground that were formed by this forced thawing work were filled with a cement-bentonite mixture four times (see Fig. 28). The total settlement of the tunnel associated with the forced thawing work was as small as 8 mm.

KAWASAKI MAN-MADE ISLAND

One of the main structures of the Kawasaki man-made island is a huge reinforced concrete diagphram wall with a total height of 119 m (with the bottom at TP –114 m), a thickness of 2.8 m and an inner diameter of 98 m (**Figs. 30** and **31**). The island was constructed as described below.

Ground improvement

The ground adjacent to, below and inside the diaphragm wall was improved, as shown in Fig. 30 and described below, before constructing the structures. This was because the existing ground comprises a very soft Holocene clay
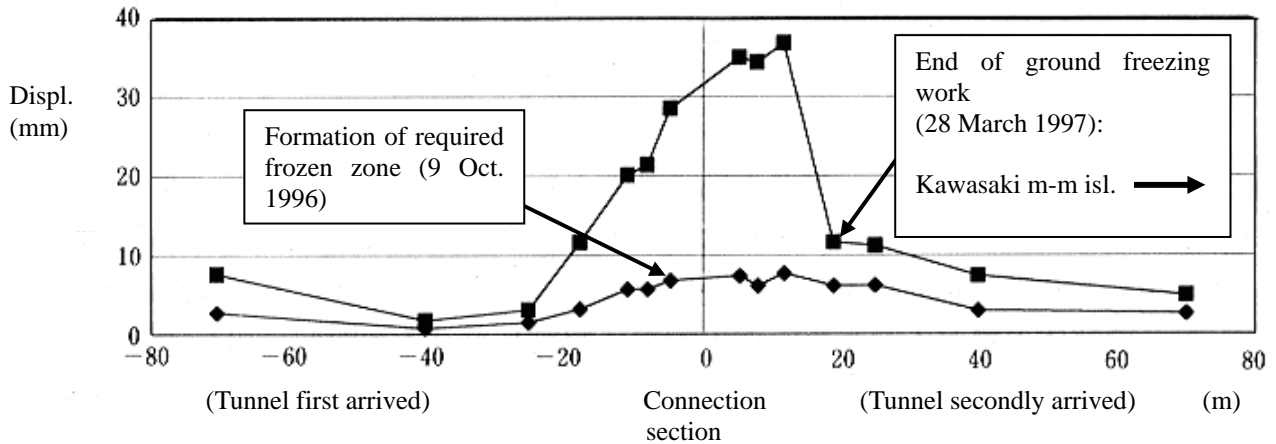
22

Figure 29: Horizontal distributions of vertical locations of the tunnel axial of the two shield machines when the planned frozen soil zone was formed and when the freezing operation was ceased around the connection section (Matsuoka et al., 1998).

deposit of about 30 m in the thickness, extending from a sea depth of TP -28 m, which could not support the island both during construction and after completion without ground improvement.

1) The clay deposit in which steel piles for the outside jacket-type steel trestle structure were to be installed was improved by the sand compaction pile (SCP) method with a sand pile diameter of 1.4 m and a replacement ratio (a_s) of 78.5 %.
2) The soft clay inside the diaphragm wall was also improved by the SCP method with a replacement ratio (a_s) of 30.1 % to ensure high trafficability of heavy vehicles for ground excavation work.
3) The soft clay deposit that was to support the diaphragm wall was improved by the ordinary type DMM with a design compressive strength equal to 18 kgf/cm^2 (1.8 MPa) at a curing time of 28 days.
4) The soft clay deposit that was to retain the shield tunnels was improved by the low strength-type DMM with a design compressive strength equal to 8 kgf/cm^2 (0.8 MPa) at a curing time of 28 days.

Construction of jacket-type steel trestle structures

After the ground improvement work was completed, the outer and inner jacket-type trestle structures were constructed, being connected to the top of the steel piles that had been driven into the ground, to serve:
1) as a mould for placing the slurry mixture that was to form the cylinder-shaped embankment;
2) as a working platform; and
3) as a shock absorber against ship impact (external jacket) during construction and after completion.
The external jacket has an external diameter of 189 m, a width at the top of 35 m and a height of 33 m. The internal jacket, which was a temporary structure and was removed before the start of excavation of the ground inside the diaphragm wall, had an external diameter of 84.5 m, a width at the top of 15 m and a height of 33 m.

Cylinder-shaped fill of cement-mixed sand slurry

The cylindrical space with a thickness of 13 m and a height of 33 m was filled with the cement-mixed sand slurry, the same as used for constructing the fill embankments at the ramp sections at the Ukishima Access and the Kisarazu man-made island. The design compressive strength was equal to 8 kgf/cm^2 (0.8 MPa) at a curing time of 28 days.

Construction of diaphragm wall

A steel-reinforced diaphragm wall, which became a water cut-off and earth retaining wall to safegurdthe RC ventilation tower structure, was constructed in the fill made of cement-mixed sand slurry and the underlying cement-mixed clay to a depth of TP -114 m below the average sea level (i.e., to a total height of 119 m). The depth of the bottom of the wall was determined as a compromise between construction period and cost and safety against heaving and boiling through the soil mass below the bottom of excavation inside the diaphragm wall. In total, 28 elements were constructed one by one by repeating a sequence comprising ground excavation, assembling of steel reinforcement cage and casting-in-place concrete. The diaphragm wall was completed in October 1992.

De-watering and ground excavation inside the diaphragm wall

Subsequently, the seawater inside the diaphragm wall was pumped out, the cement-mixed sand fill remaining along the inside face of the diaphragm wall was removed, and the internal jacket was progressively withdrawn. The sea bottom of the Tokyo Bay at a depth of about 28 m was exposed for the first time in April 1993.
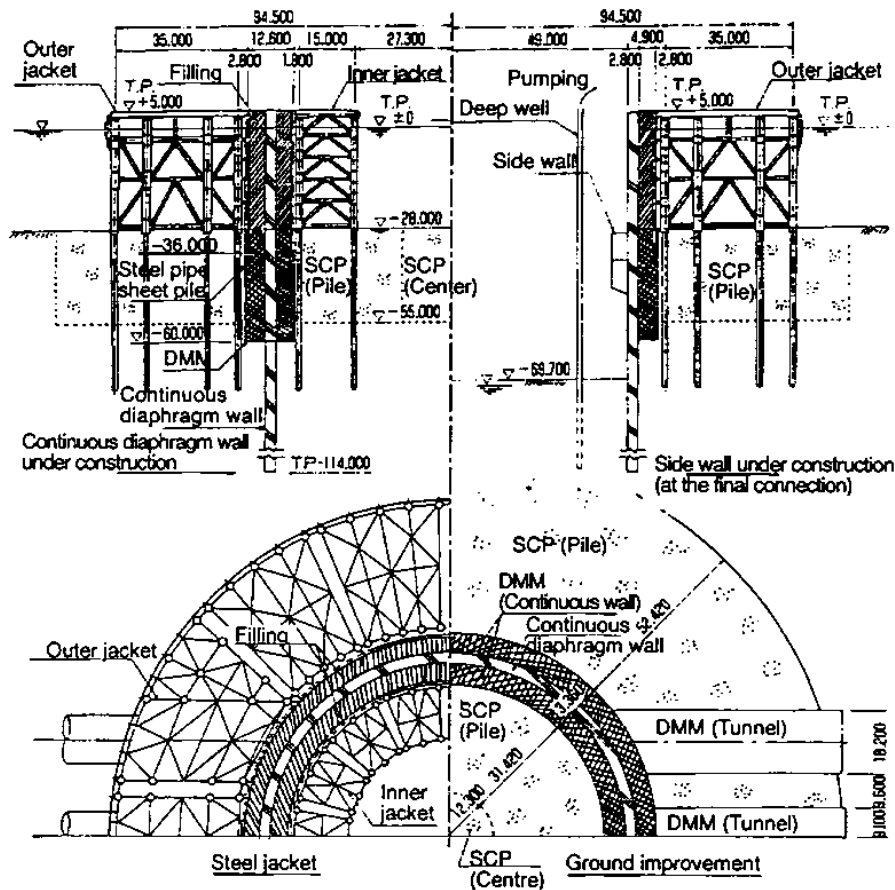
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Figure 30:   Construction procedure of the Kawasaki man-made island and associated ground improvement work.

It is standard practice with excavation work under dry conditions inside a diaphragm wall that either the groundwater level in the adjacent ground is made sufficiently low by means of deep wells, or the bottom end of the diaphragm wall is installed sufficiently deep into a low-permeability soil layer (if available), or both.   In the present case, the water pressure difference between the final excavated ground surface inside the diaphragm wall and the outside of the diaphragm wall is about 0.7 MPa, which is too large for the stability of the soil mass inside the diaphragm wall.   It was therefore necessary to lower the groundwater level in the adjacent ground to a sufficiently low level.   It was decided to take advantage of a thin low-permeability clay layer (D5c1; **Fig. 32b**), with an average thickness of only 65- 85 cm at a depth of TP -111 m and 3 m above the bottom of the diaphragm wall, and a thicker low-permeability clay layer (D5c2), with a thickness of about 3- 4 m at a depth of TP -125 m from the sea level and 14 m below the bottom of the diaphragm wall.   The soil layers above, in between and below the two clay layers are basically of fine sand, having a relatively high permeability.   Before starting the ground excavation inside the diaphragm wall, to avoid heaving and boiling through the soil mass below the bottom of excavation inside the diaphragm wall, twenty four deep wells with a diameter of 60 cm were installed, with the deepest one to a depth of TP –168.5 m from the sea level (**Fig. 32a**).   The groundwater level in these sand layers (named layers A, B and C) were decreased as listed in Table 2.

Table 2:   Ground water levels in the drainage wells and the ground inside the diaphragm wall, immediately before the seepage accident, the Kawasaki man-made island (Ouchi et al., 1998).

	Ground water level in the drainage wells (TP)	Ground water level observed in the soil layer within the diameter of the diaphragm wall (TP)
Layer A	- 74.0 m	- 73.0 m
Layer B	- 80.0 m	- 65.0 m
Layer C	- 42.0 m	- 40.0 m

About 10,000 tons per day of groundwater was pumped up before the start of excavation of the ground inside
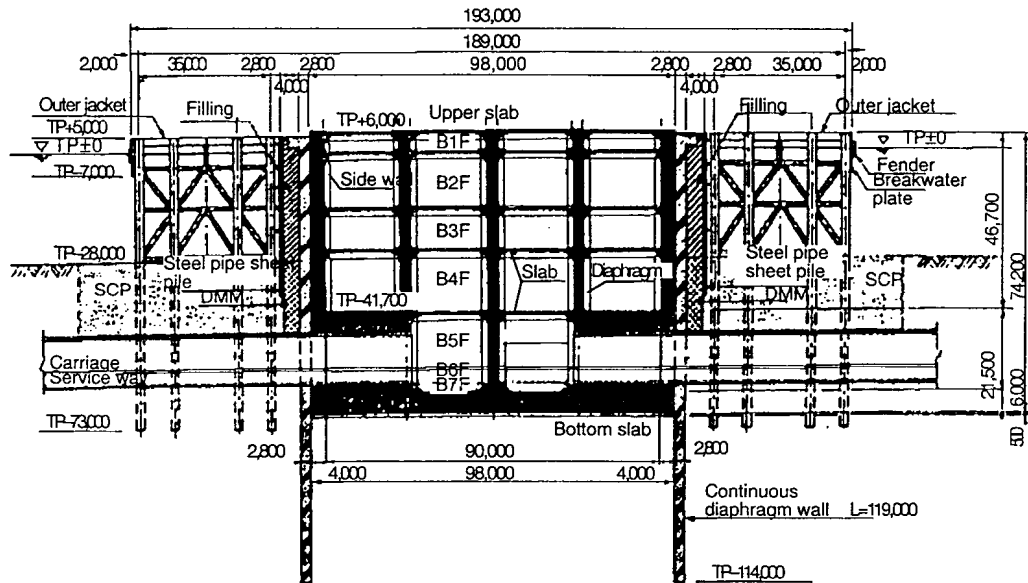
Figure 31: Completed Kawasaki man-made island.

the diagphram. The water pressures at key locations were carefully monitored. The ground inside the diaphragm wall was excavated to a depth of 42 m from the sea bottom (a depth of about TP -70 m from the average sea level). **Fig. 33** shows typical pore pressure distributions, lateral displacements of diaphragm wall and circumferential stresses in the reinforcing steel bars of the diaphragm wall measured at the end of excavation work when the groundwater behaviour was normal. At the end of excavation around the end of September 1993, the amount of groundwater pumped out was about 200 m^3 per day from layer A, while it was 1,200 m^3 per day from layer B and about 8,500 m^3 per day from layer C, which was about half of the respective predicted value.

Accidental groundwater spouting from the ground inside the diaphragm wall

The completion of ground excavation work was celebrated on 26th October 1993. After having placed a 0.5 m-thick base unreinforced concrete layer and installed four blocks of the ten steel frameworks for the inside RC structure, on 14 (Sunday) November 1993, spouting of groundwater with an unusual amount, much more than usual, started from the ground inside the diaphragm wall. Groundwater spouting was from a PVC pipe with a diameter of about 10 cm, placed along the west part of the inside wall face of the diaphragm. The bottom end of the pipe reached a gravel layer that had been placed between the excavated ground and the base concrete layer.

The following is the sequence of events.
1) The water erupted from the PVC pipe was fresh water, not salt water as the one from layer A. So, to reduce the amount of groundwater spouting, the groundwater level in the drainage well to layer C was lowered, from TP- 42 m to TP- 50 m, which was the maximum allowable value prescribed not to cause lateral compressive buckling failure of the drainage wells by surrounding high water pressure.
2) As a result, the increase in the rate of groundwater spouting stopped after it had become about 2,000 tons per day (see **Fig. 34**). It was considered that this amount of water spouting could be dealt with by making a pond around the outlet of the pipe (to decease the hydraulic gradient) and increasing the amount of pumped-up groundwater.
3) Subsequently, however, the amount of groundwater spouting increased day by day, which had become about 3 tons per minute or 4,500 tonf per day on 24th November, indicating a danger of heaving and/or boiling failure in the ground inside the diagphram wall. It was considered at that time that if it would take place, it would endanger and maybe kill about 3,000 engineers and technicians working inside the diaphragm wall, and if this accident would take place, it would destroy the Kawasaki man-made island, resulting into a fatal accident for the construction of the highway. Indeed, this is the worst scenario that the engineers considered in advance.
4) Then, it was decided to pour back inside the diaphragm wall the groundwater that had been pumped up, which was fresh water, to decrease the hydraulic gradient in the ground inside the diagphram wall. The water became deeper by one meter per day, which resulted to a depth more than six meters on 30th November.
5) It was soon found however that this amount of water was not enough to stop the high rate of spouting of groundwater from the ground inside the diagphram wall. Then, on 2nd December, a great decision was made to pour sea water from the outside of the island into the inside of the diaphragm wall until the rate of groundwater spouting became small enough. The depth of water became nearly 16 m (i.e., around TP -50 m) 17 hours after having started pouring sea water. It was on 3rd December when the rate of groundwater spouting returned to the original small

value.

6) Keeping the hydraulic gradient small by maintaining a high water level with a depth of about 16 m inside the diaphragm wall, the portions of ground through which the groundwater was seeping were improved by grouting a cement-bentonite mixture with a total amount of 550 m^3, then a solible silicate with a total amount of 400 m^3. The grouting was continued until February 1994. The existing wells to layer C were reinforced by inserting an inner sheath pipe in each well.

7) It was then decided to decrease the hydraulic gradient between layer A and layers B and C inside the diameter of the diaphragm wall. The groundwater levels in layers B and C were lowered as close as possible to that in layer A (i.e., TP –70 m). Based on results of numerical analysis, the number of drainage well inside the diameter of the diaphragm wall was increased by six to layer B and by four to layer C.

8) After having improved the deep well system, on 7th March 1994, pumping out of water from the inside of the diaphragm wall was started. On 8th April, the base concrete appeared again after a period of four and half months.

9) Reinforcement steel members, which had been submerged in seawater, were then washed with fresh water, and mud that had been accumulated inside the diagphram wall was removed. A bottom RC slab, which was 6 m-thick with a total weight of 100,000 tonf, was then cast-in-place.

10) By this accident, the start of shield tunnelling work was delayed about six months. The direct extra cost of this accident amounted to 2 billion yen.

Inferred causes for the seepage accident

It was inferred that the spouted groundwater came from layer C for the following reasons:

a) The quality of spouted water was similar to that of the water pumped from layers B and C, while dissimilar to that from layer A, which includes a larger salt content.

b) The amount of pumped water from layer C, with a controlled fixed groundwater level of TP- 42 m in the drainage pipe, started decreasing according to the water spouting from a PVC pipe and later from cracks in the base concrete layer.

It was attempted to identify the cause for the accident. The following compound mechanisms had been considered to be relevant (**Fig. 35**):
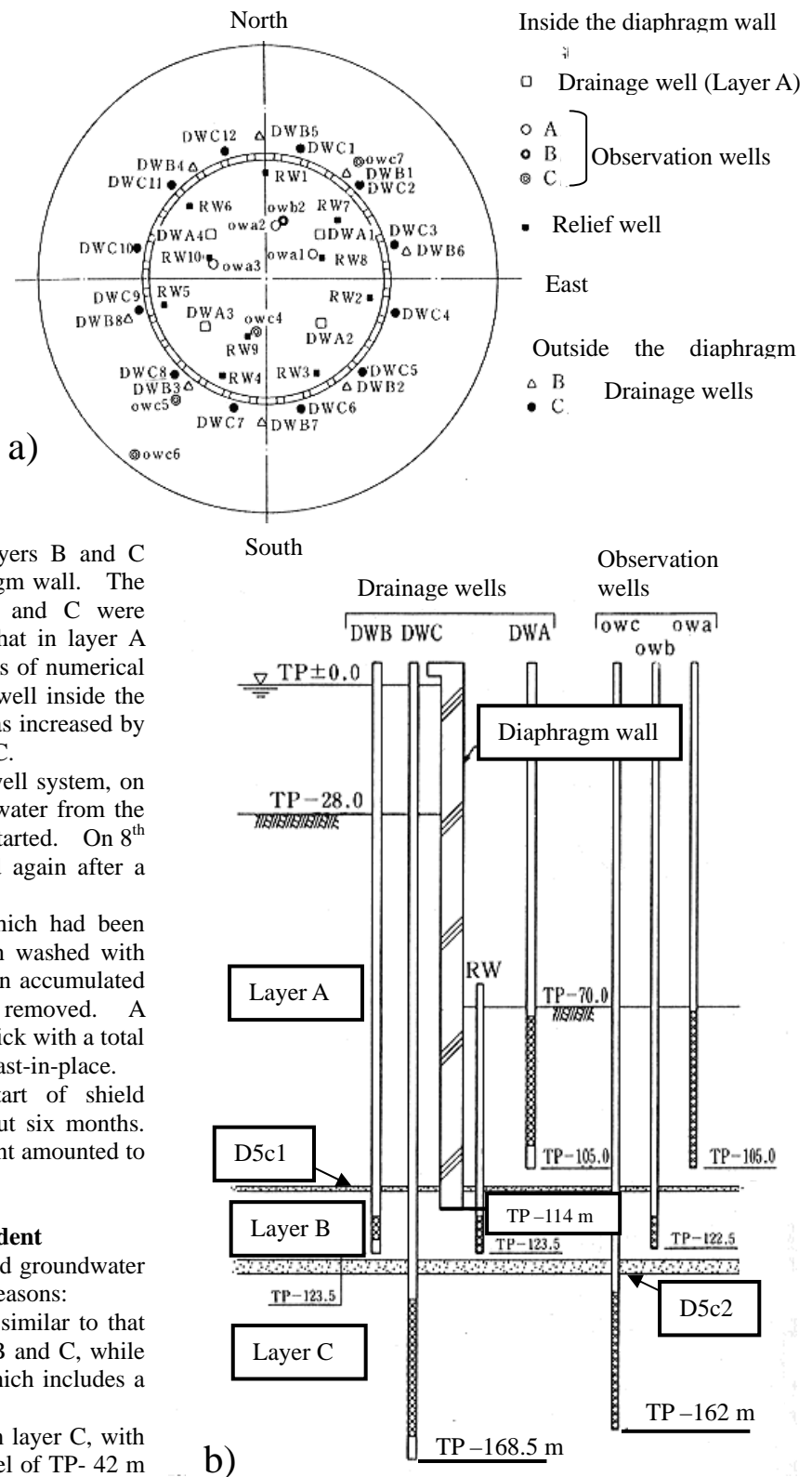
1) The lower clay layer D5c2 was damaged by

Figure 32: Locations of drainage, measurement and relief wells at the Kawasaki man-made island (Ouchi et al., 1998)

the following three factors:

a) The observed head difference between layers B and C in the ground within the diameter of the diaphragm wall was too large, resulting in a too large hydraulic gradient (see Table 2).

b) The thickness of D5c2 layer at the place through which an excessive amount of groundwater seepage took place was found to be less than 2 m, which was much less than the values estimated in advance (4.4 m and 2.8 m in the east and west sides).

c) The deformation of the ground inside the diameter of the diaphragm wall due to heaving by removing the weight of soil by excavation inside the diameter of the diaphragm wall and by applying the weight of the diaphragm wall was larger than estimated.

2) The ground adjacent to the drainage, observation and relief wells and inclinometers for measuring the displacement of the diaphragm walls that had been installed in the ground inside the diameter of the diaphragm wall may have been loosened during their installation, having become very permeable.

3) The thickness of layer D5c1 was only 65- 85 cm.   This clay layer may have been damaged by the construction of the diaphragm wall, having become very permeable, and also partly by continuous seepage from outside the diaphragm wall towards the inside.
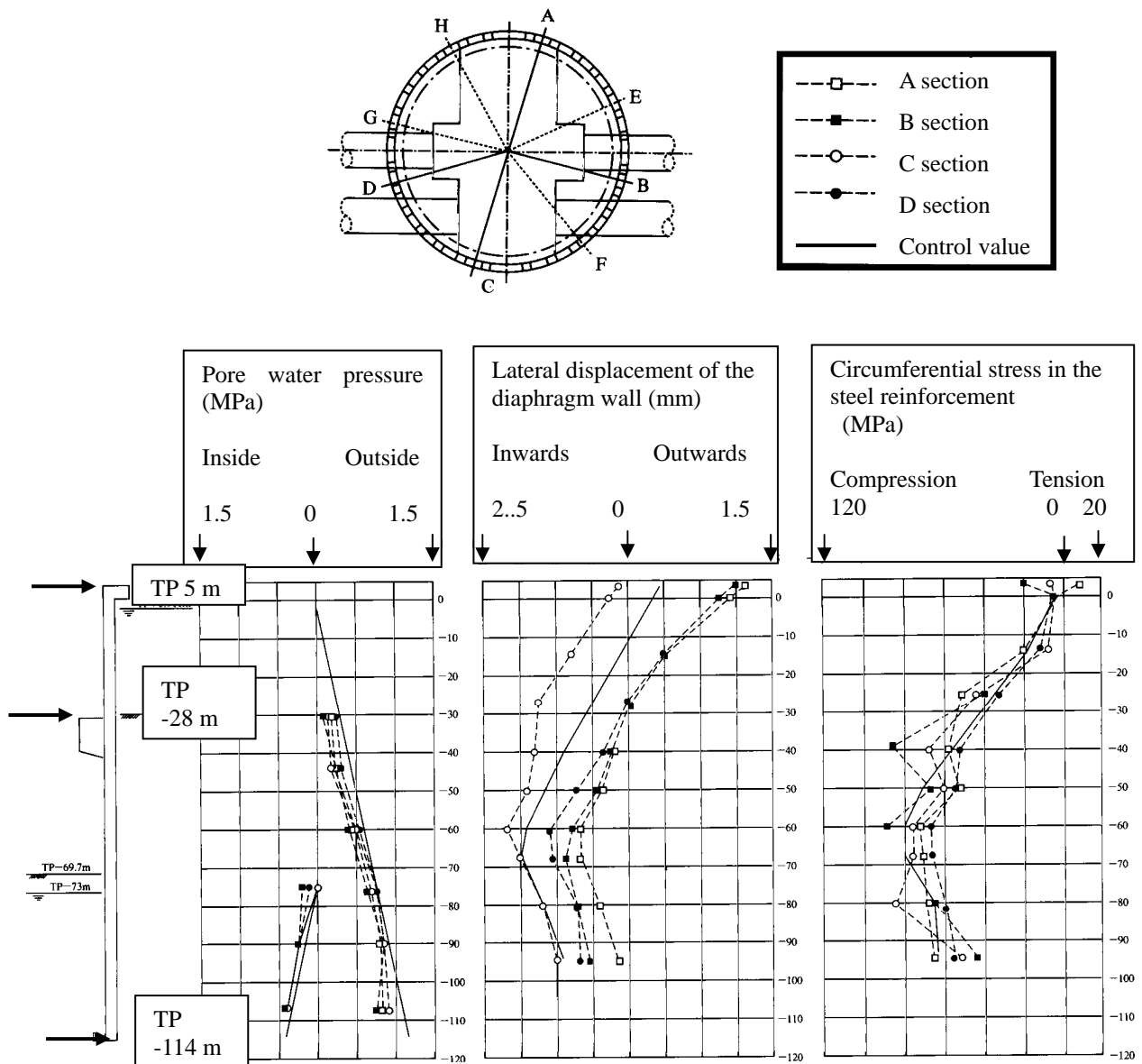

Figure 33:   Typical measured pore pressure distributions, lateral wall displacements and circumferential stresses in the reinforcing steel bars acting at the diaphragm wall, Kawasaki man-made island (JSCE et al., 1996).
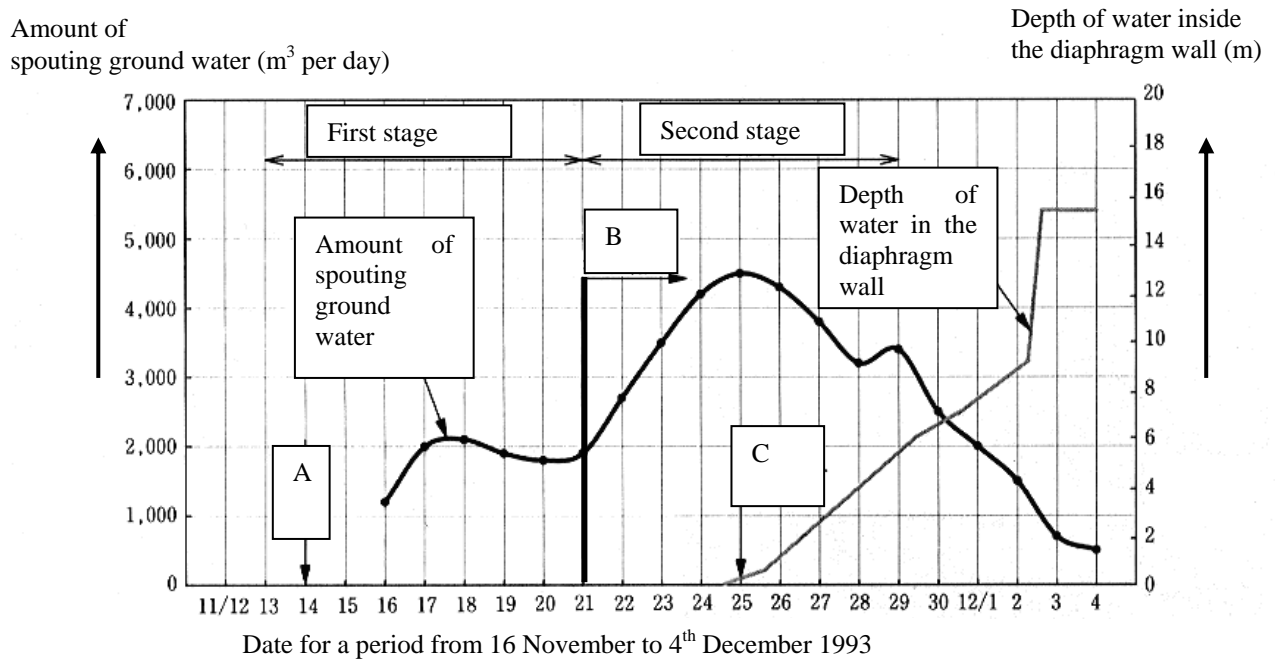
Amount of
spouting ground water (m³ per day)

Depth of water inside
the diaphragm wall (m)

Date for a period from 16 November to 4th December 1993

Figure 34:   Time histories of the rate of ground water spouting (m³ per day) and the depth of water inside the diaphragm wall (m), Kawasaki man-made island (Ouchi et al., 1998):
>   A: first observation of unusual ground water spouting from a PVC pipe.
>   B: the rate of ground water spouting was increasing for this period.
>   C: start of pouring water to the inside of the diaphragm wall.

Some engineers of the contractors in charge of the construction of the Kawasaki man-made island considered that a more ample safety factor against the ground failure by heaving and boiling could have been ensured if the following measures had been taken (Nikkei Construction, 1997):
a) The bottom of the diaphragm wall was made deeper than the actual one, deeper than layer D5c-2, even if the extended part of diaphragm wall was not steel-reinforced.   By this measure, it would have become easier to lower the groundwater level in layers B and C (in particular that in layer C) inside and immediately below the diaphragm wall and thereby to decrease the hydraulic gradient, reducing the danger of heaving and boiling;
b) The hydraulic gradient between layers A, B and C was made smaller than the original design value.   This measure would have become feasible by the measure described above.
c) A larger number of deep wells was used to ensure lower groundwater levels in layers B and C.

CONCLUDING REMARKS

The Trans-Tokyo Bay Highway was constructed in a relatively deep sea, crossing heavy shipping routes, under poor ground conditions with a high level of seismicity.   For this reason, several design and construction problems of geotechnical engineering had to be solved for the success of the project.   The following lessons were obtained:
1) The use of four types of ground improvement by means of cement-mixing played an essential role to solve many of the potential problems.
   a) To evaluate the safety factor for the ultimate failure state of a large scale cement-mixed mass under water, the limit equilibrium-based stability analysis using the residual shear strength was used, instead of the conventional allowable stress method.   In this design, the effects of initial effective confining pressure, hydrostatic pressure and drainage conditions were taken into account.   This design method was relevant for the TTB Highway project and will be for similar projects.
   b) A method for constructing fill embankments underwater using a slurry of cement-mixed sand, controlling the strength, was developed.   It was shown that the selection of relevant mixing proportion and the development of relevant underwater placement method using tremmie pipes were among the important keys for the success.
   c) The conventional deep mixing method to improve existing soft clay deposits by in-situ mixing with a cement slurry was modified by reducing the amount of cement while increasing the water/cement ratio to obtain a

homogeneous cement-mixed clay having a specified strength.

d) A large submerged fill embankment having a sufficiently high strength was constructed for a relatively short construction period by dumping underwater a dry mixture of cement-mixed sand. The selection of relevant mixing proportion and the development of a special double-tube chute were among the keys for the success.

2) Long shield tunnels with the world's largest diameter were constructed successfully. Several key issues are as follows:

a) By using relevant cutting blades with careful monitoring and control of cutting torque and excavation speed, smooth shield tunnelling in successive stiff cement-mixed soil and unimproved very soft clay became feasible. In particular, the mud slurry pressure was very carefully controlled to prevent active and passive failure of unimproved very soft clay in front of the cutter head of shield machine.

b) The possible floatation of tunnels when proceeding in an unimproved soft Holocene clay at a very shallow depth was prevented by using temporary weight with monitoring the stability of the overlying soil layer.

c) The shield tunnel drive was shortened by underground connection of shield machines with the help of the ground freezing technique.

3) A huge offshore diaphragm wall of 98 m in internal diameter and 119 m in total height was constructed. A serious seepage accident took place in the ground inside the diaphragm wall immediately after the ground excavation work inside the diaphragm wall was completed. The completion of the construction project was delayed about six months.

a) This accident could have become fatal for the success of the project if relevant measures were not taken promptly.

b) The accident might have not taken place if the hydraulic gradient in the ground inside and immediately below the diaphragm wall had been substantially lower. One of the effective methods for the above would have been to make the bottom of the diaphragm wall substantially deeper than the actual one.
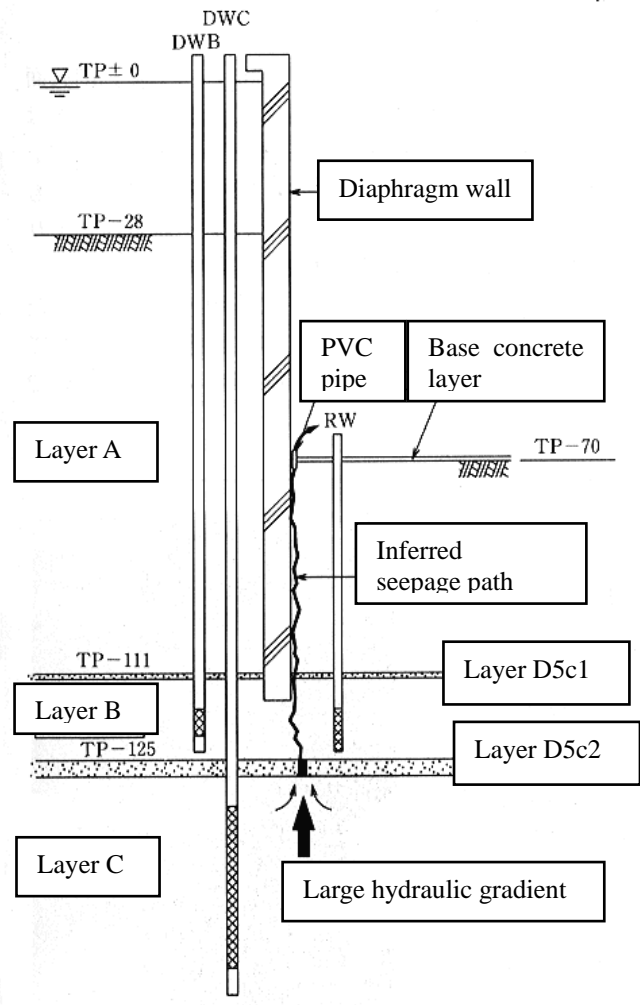

Figure 35: Schematic diagram illustrating the inferred mechanism of ground water seepage accident inside the diaphragm wall, Kawasaki man-made island (Ouchi et al., 1998).

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